

HYDRAULIC GEOMETRY ANALYSIS OF THE LOWER MISSISSIPPI

Final Report

by

Philip J. Soar

JBA Consulting
Magna House, South Street
Atherstone CV9 1DF, UK
Tel. +44 (0) 1827 722710
Fax. +44 (0) 1827 722719

Colin R. Thorne and Oliver P. Harmar

School of Geography
University of Nottingham
Nottingham NG7 2RD, UK
Tel. 0115-951-5431
Fax. 0115-951-5248

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BACKGROUND

The hydraulic geometry of the Lower Mississippi River is primarily the product of the action of natural flows acting on the floodplain materials over centuries and millennia to form an alluvial forming a channel. However, the modern channel has been modified by engineered channel improvements performed as part of the Mississippi River and Tributaries (MR&T) project. Channel improvement features include meander cutoffs, levees, diversion structures, bank stabilization, and low training dikes, which have been constructed to enhance flood control, stabilize the channel, and provide sufficient depth for navigation even at times of low water. The shape and form of the modern channel have been further influenced by morphological responses to artificial stabilization and training. Historically, morphological response involved bank erosion, siltation and some recovery of sinuosity lost due to artificial cutoffs, though more recently bank erosion, channel migration and planform evolution have all but ceased as ACM revetments now extend along a great proportion of the banklines. Substantive legislation for the MR&T project, and the initiation of the construction program, began following the catastrophic flood of 1927, and this event marks the start of the modern era in terms of channel form and process in the Lower Mississippi River. Since that time, channel dimensions and geometries have evolved under the influence of natural flows, direct engineering interventions and dynamic adjustment driven by process-response feedback loops linking flow hydraulics and channel morphology.

Previous studies have documented the stage and slope adjustments on the Lower Mississippi River, but the associated changes in hydraulic geometry have not yet been fully investigated. In principle, hydraulic geometry analysis could be undertaken using comprehensive and annual hydrographic surveys of the river. While there is no doubt that comprehensive hydrographic surveys provide valuable information with respect to long-term channel changes, evaluation of the comprehensive surveys alone would be of limited value because they represent snapshots of channel form at approximately ten year intervals. In fact, it is known that annual fluctuations in channel geometry may be greater than the changes observed between surveys taken ten or more years apart. For this reason it is important to investigate changes in channel form based on analysis of annual hydrographic surveys to elucidate the type and degree of changes to be expected in the short-term.

Only by combining knowledge of short and long-term changes will it be possible to separate fluctuations in hydraulic geometry from wider-scale adjustments of the fluvial system. While both types of change are important, it is only the latter that represent the progressive response of the river to natural flows, engineering interventions and the operation of complex process-response.

Research on sediment dynamics and channel changes in the Lower Mississippi River has been performed at the University of Nottingham under two previous research grants from the US Army Research Group (London). The first was titled 'Sediment Transport in the

Lower Mississippi River' (Contract number: N68171-00-M-5982, Project number: W90C2K-9016-ENO1). This initiative concerned compilation of sediment transport records from data gathered at USACE and USGS gauging stations along the Lower Mississippi. In a continuation of this work, the project titled, Morphodynamics of the Lower Mississippi River'. This report contains initial statistical analyses of the sediment transport records and preliminary interpretations of the morphological implications of records that document changes in sediment fluxes at the decadal scale.

Following completion of the work funded by the US Research Office, the University of Nottingham awarded a University Research Studentship to Mr Oliver Harmar. He chose as his doctoral topic further research on the Lower Mississippi utilising the dataset compiled at Nottingham plus additional data to be obtained from the relevant USACE District and Division offices. Most specifically, he has obtained and worked up annual hydrographic surveys of the river in the Vicksburg District covering approximately 60 miles of the river between RM 560 and 620. Nine surveys are available, between 1992 and 2001. The morphological information is matched by continuous hydrologic data on discharges and water levels from gaging stations and high quality maps and aerial photographs of the river.

The work performed in this project builds on the earlier studies by using the previously compiled data set for RM 560 and 620 as the basis for detailed analysis of channel geometry and its variation in time and space.

CONTEXT

Lower Mississippi River Channel Geometry

The Lower Mississippi River flows downstream from the confluence of the Ohio River at Cairo, Illinois to the Gulf of Mexico at the Head of Passes, Louisiana. Along this course, the river flows in a meandering planform, first through its alluvial valley and further downstream across its deltaic plain. Prior to extensive engineering modification as part of the Mississippi River and Tributaries (MR&T) Project, the Lower Mississippi River was free to adjust its energy gradient through a combination of gradual evolution and avulsive changes to its planform geometry. This was achieved through incremental growth and, periodic cutoff of meander bends due to high rates of bank erosion and lateral channel mobility (Schumm *et al.*, 1994; Hudson and Kesel, 2000). In its natural state, Winkley (1977) reports that meander bend cutoffs developed quickly though not instantaneously, and following a cutoff, the river took between 30 and 80 years to regain its pre-cutoff width and bar sequencing. Moreover, cutoffs did not occur in multiples, but rather a long term balance existed between reaches that were shortened by cutoffs and those simultaneously being lengthened by meander growth.

During the twentieth century the geomorphology of the Lower Mississippi River has been influenced by a series of engineering modifications to improve flood control and aid navigation. Channel slope was artificially steepened, and sinuosity reduced, by the construction of fourteen artificial cutoffs during the period 1932-1942 in the reach between Memphis and Old River distributary. Following the cutoff period, bank stabilization was undertaken along most of the length of the river using articulated concrete mattress and upper bank riprap. The effect has been to fix the relatively steep, low sinuosity post-modification morphology of river into a near-permanent planform alignment. This is significant geomorphologically because the Lower Mississippi River can no longer adjust its energy slope through planform adjustments, but instead is, instead, restricted to making adjustments to its longitudinal profile. MR&T work, as the name suggests, included measures on the tributaries as well as the Lower Mississippi

itself. Improved land management practices in the watershed, together with construction of large reservoirs and channel improvement works on the Missouri and other large tributaries are believed by some researchers to have led to a dramatic reduction in suspended sediment load supplied to the Lower Mississippi. Reduction of upstream supply, coupled with the virtual elimination of sediment input from bank erosion seems to have resulted in a mark reduction in the amount of sediment transported by the Lower Mississippi River (Keown *et al.*, 1981; Robbins, 1977; Kesel, 1989).

Recognition is growing that the geomorphological drivers and behaviour of the Lower Mississippi River has changed as a result of these engineering modifications. Accepting that geomorphological change, partly natural and partly in response to engineering interventions, is occurring leads inevitably to the conclusion that improved longer-term and shorter-term channel management strategies require an accurate understanding of how the river changed during the twentieth century.

Previous researchers have inferred channel morphological adjustments from temporal variations in specific gauge records at gauging stations along the Lower Mississippi River. Specific gauge records are plots of flow stage through time for a specific discharge. The discharge chosen is usually near bankfull because this has been shown to consistently approximate the dominant discharge (Wolman and Miller, 1960; Hey, 1982). Winkley (1977) concludes from this form of analysis that, in the pre-cutoff period, the Lower Mississippi River was a stable system between Cairo, IL and Natchez, MS. In the post cut-off period, subsequent analysis of specific gauge records from Cairo downstream to the Old River distributary between 1950 and 1994 has been used to propose a pattern of reach-scale 'morphological response' in the longitudinal profile (Biedenharn and Watson, 1997; Figure 1).

Upstream of Arkansas City, specific gauge records suggest a degradational trend, while downstream of Vicksburg the records suggest an aggradational trend. In the reach between Arkansas City and Vicksburg, the absence of a significant trend has been used to propose a that a 'hinge zone' of relative stability exists. Hence, this model presents

evidence to suggest that at the reach scale, the river is responding to restore a stable condition in a manner broadly similar to the type of morphological response associated with a single cutoff (as outlined by Lane, 1947).

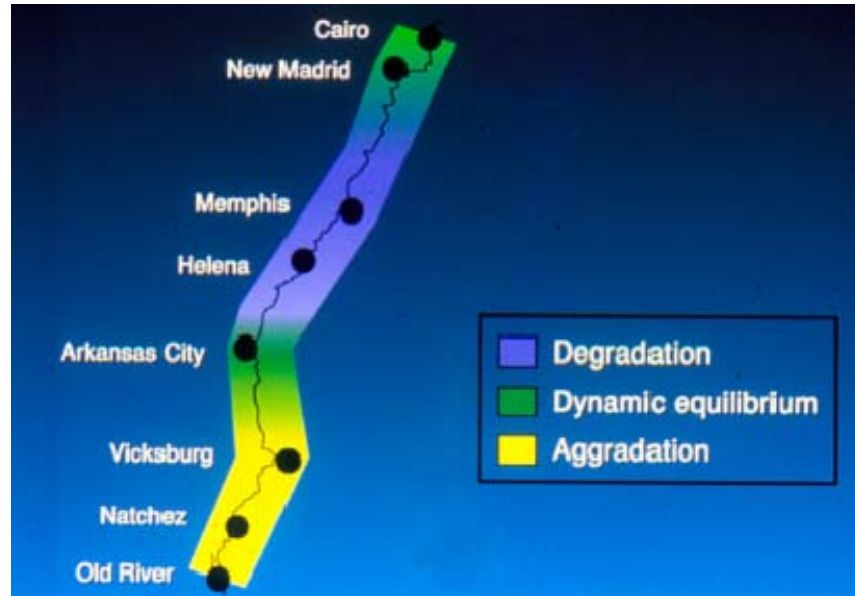


Figure 1. Regional scale adjustments to the longitudinal profile in the period 1950-94, proposed by Biedenharn and Watson (1997) from analysis of specific gauge records.

This proposed model of response provides a useful overview of geomorphological response but its utility for planning future river management is limited for two reasons. First, while specific gauge records reveal spatial and temporal variations in the water surface, these do not necessarily represent changes at the channel bed because changes in flow resistance are unaccounted for. Second, the simplicity of the model inevitably masks important details of reach and sub-reach scale morphological behaviour. Further, examining records for the pre-modification river, Schumm *et al.* (1994) describe the importance of neotectonic activity and geological factors in influencing morphological behaviour at the reach and sub-reach scales. However, no study has investigated the continued importance of such influences on morphological behaviour in the post-modification period.

Logically, analysis of hydrographic survey data sets collected during the late 19th and 20th centuries should be the next stage in improving understanding of long term geomorphological response behaviour, because it allows analysis of morphological changes directly through examination of the bed topography, rather than indirectly through gauge records. Also, analysis at much finer spatial and temporal scales than specific gauge records is possible as the hydrographic data coverage is almost continuous. It follows that the viability of revealing spatial and temporal changes in channel geometry using hydrographic survey data should be explored in depth as a potentially key resource in developing improved river management strategies and works for the 21st century.

DATA ASSIMILATION

Data Acquisition

Study Reach

The study reach is located between river miles 565 and 618. The nearest gaging stations are Arkansas City at river mile 554.1 RM, Rosedale at river mile 592.2 and Helena at river mile 663.1. The reach includes the confluences of the White and Arkansas Rivers with the Mississippi River.

Base data were obtained from doctoral work being undertaken at the University of Nottingham by Oliver Harmar. Pre-processing consisted of extracting cross-sections from the annual hydrographic survey records supplied to him by the Vicksburg District USACE.

Low Water Reference Plane

The Low Water Reference Plane (LWRP) on the Mississippi River is used to establish the crown elevation for dikes and other engineering works. It is also used by navigation interests to obtain a general understanding of the depth of water available at critical locations in the river.

The LWRP elevations along the Mississippi River are developed from LWRP stages computed at individual gauging stations based on the 97% exceedance flow for a specified period of record (typically from 1954 to the time of computation) being applied to a series of rating curves from a more recent period (typically the past 10 years). The LWRP was most recently computed in 1992 using the 1954 to 1991 period of record flows and 1982 to 1991 rating curves. The procedure for deriving the LWRP profile is:

- i) Compute the 97 percent exceedance flow at each of the discharge gaging stations.
- ii) Develop a set of rating curves (one for each year from 1982 to 1991). This is typically fitted by eye.
- iii) Convert the 97 percent exceedance flow to stages on each of the rating curves and taking the average of the 10 stages to determine the LWRP elevation.
- iv) Use documented low flow profiles (particularly that of October 1991) to adjust the elevation of the LWRP between the gauging stations.

Information specific to the 1993 LWRP (to be used as the basis for measuring channel geometry in this study) was obtained through a meeting held at ERDC in Vicksburg in June 2003. Mr Rick Robertson was present at the meeting and he was able to give a full account of how the 1993 LWRP was constructed.

The procedure used discharge data for Arkansas City gauging station from 1954 to 1991 to derive a flow duration curve. LWRP is associated with the discharge that is equaled or exceeded 97% of the time, taken from that flow duration curve. The discharge so obtained was 166,000 cfs. To determine the elevation of LWRP at the Arkansas City gauge requires a stage-discharge relationship. Ten annual rating curves for the period 1982 to 1991 were derived using observed discharges and stages. The rating curves were fitted by eye and, where necessary, extrapolated down to the LWRP discharge. The stage corresponding to the 97% accident flow was read from the stage-discharge curve for each year and the average was taken to define the elevation of the 1993 LWRP at Arkansas City. The resulting stage was -1.1 ft, corresponding to an elevation of 95.56 ft (NGVD). Mr Robertson kindly supplied copies of all relevant documentation pertaining to his calculations.

The longitudinal profile of the 1993 LWRP was determined using observed water surface profiles for very low flows in 1988 and 1991. LWRP was adjusted to plot between these two water surface profiles in a consistent manner. At the upstream end of the Vicksburg District, steps were taken to tie in the LWRP with that for the Memphis District (based on the Helena Gage). The relevant data for Helena are: $Q = 153,000$ cfs, stage = -2.2 ft, elevation = 139.5 ft (NGVD).

Other matters concerning LWRP that were clarified at the meeting included reasons for selection of 1954 as the start date for hydrological records used to derive the 97% flow (probably related to the completion of reservoirs on tributaries that affect the low flow regime of the Mississippi); and how best to deal with discharge introduced by the Arkansas and White Rivers if it was desired to relate stages above LWRP to discharges at individual cross-sections between RM 560 and 620.

Separation of bends and crossings

In a meeting held at EDRC between the contractors and staff from the Vicksburg Division USACE, it was pointed out by Mr. John Brooks that hydraulic geometry of crossings may differ significantly from that of meander bends. It was agreed that (as per the initial proposal) that a channel planform curvature parameter would be used to discriminate between bends and crossings, with separate channel geometry analyses being performed for each.

Divided Channels

It was pointed out by Mr. Don Williams that the channel geometry of divided reaches with secondary channels would be different to that of single-thread reaches. Given the ecological importance of secondary channels, information on the geometry of such reaches would be of interest. It was agreed that care would be taken to identify divided cross-sections and treat them separately to explore the possibility of defining unique equations for these reaches.

Engineering features

Mr. Steve Scott notified the contractors about recently constructed engineering structures in the study reach that might influence hydraulic geometry in particular ways:

Bendway weirs built at Victoria Bend (1995) – not operating as intended.

Works at mouth of White River (2001)

Break out of Arkansas River in 1997

Arkansas River mouth to Miles Landing – has been very stable.

Data Screening and Pre-processing

1993 LWRP

The LWRP dataset compiled by the Vicksburg District comprises a text file of River Miles and corresponding Elevations (ft). The extent of this dataset covers approximately 310 miles, as shown in Figure 2 (which also includes data covering approximately 16 miles above river mile 592 computed by the Memphis District). From this dataset, the LWRP for each cross-section in the annual surveys (between river miles 565 and 618) was derived by linear interpolation.

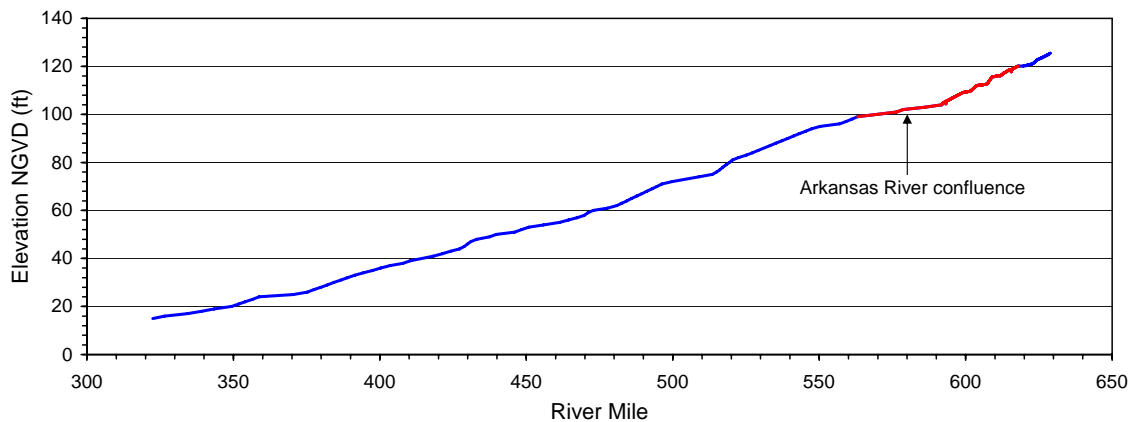


Figure 2. LWRP elevations in the Lower Mississippi computed by the Vicksburg District (including elevations computed by the Memphis District above RM 592). Red reach delimits the extent of the profile used in this study.

Pre-processing procedure for hydrographic survey files

Processing Steps

The majority of the pre-processing of the hydrographic survey data was undertaken at the University of Nottingham under work led by Oliver Harnar. The objective of the pre-processing stage was to develop a series of automated routines for extracting survey data pertaining to individual cross-sections as stand alone text files from the full data set of each annual survey, originally compiled in GIS format.

Each annual hydrographic survey file for the period 1992-2001 was initially obtained in Microstation design file format (.dgn) from the USACE Vicksburg District Office, Vicksburg, MS. These are CAD-compatible files in which information is stored in multiple layers. Four stages of data set pre-processing were applied, as summarized below:

a) Extraction of xyz hydrographic coordinates into a standard text file

The first stage of data pre-processing involved isolating the hydrographic point data (x, y and z coordinates) from other information stored within each file and saving the extracted data in standard text files. This was undertaken using ESRI Arc View 3.3 GIS.

b) Identification of individual cross-sections

The second stage involved developing an automated procedure capable of sorting the tabulated series of x, y and z coordinates within each hydrographic survey data file into a series of unique cross-sections.

Prior to the application of this procedure, cross-sections which did not lie perpendicular to the channel centreline were manually removed from each survey. These 'problem' cross-sections were found to intersect other cross-sections and hence, would have generated errors in the processing procedure. Cross-section delineation is based on calculating the distance and angle between consecutive observations in the unstructured table of coordinates. Points were incorporated into the same cross-section if the distance between successive points is less than 65 m and if the angle between successive points does not differ by greater than 2.5 degrees from the angle made by the preceding pair of points. Where a series of consecutive observations have been grouped into a single cross-section, but the next consecutive observation exceeds both the distance and angle thresholds, all of the other observations in the table are searched. A further observation is added to the cross-section only if it satisfies both the distance and angle thresholds, and also if linear regression analysis of the generated cross-section derives a coefficient of determination (R^2) value exceeding 0.95.

Following application of the delineation routine, two more data quality checks were performed to better validate the dataset: calculation of the maximum distance between any two observations in each cross-section and; calculation of the standard deviation of the distances between observations in each cross-section. Anomalies identified at this final stage of the data quality checking procedures were further inspected.

c) Sorting of cross-sections into a downstream order

The orders of points and cross-sections were rearranged such that successive points per cross-section are from the right bank to the left bank (facing downstream) and successive cross-sections are in the downstream direction. This routine involved a series of macros, and involved: i) finding the centre-point and the furthest left and right bank observations

for each cross-section; ii) sorting the centre-points so that they are ordered in a downstream direction; iii) using the ordered list of centre-points to sort each cross-section into the correct downstream order, and; iv) sorting points within each cross-section so that they are ordered from the right bank to the left bank in the downstream direction

A final visual inspection was undertaken following application of the sorting routines. This involved displaying the computed centre-point and associated left and right bank coordinates for each cross-section in a GIS layer (see Figure 3).

d) Rectifying the cross-section points onto straight lines

The final stage involved minor displacement of the cross-section points onto straight lines drawn between the two end points. This was necessary for calculating some of the cross-section parameters (e.g. wetted perimeter, cross-section area) in the channel geometry analysis developed herein.

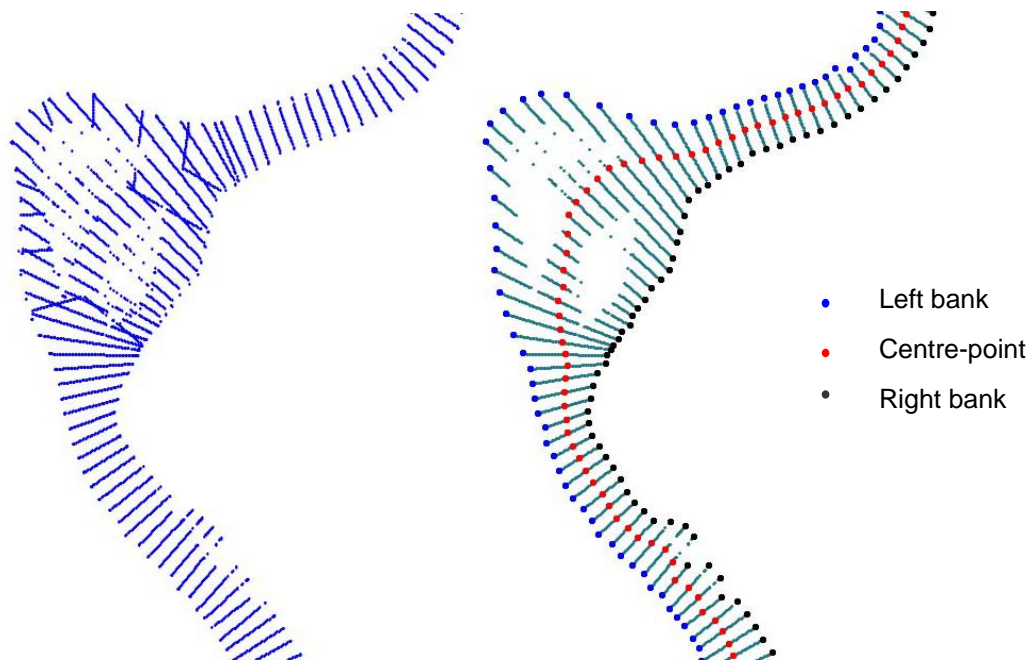


Figure 3. Example of initial data processing: original cross-sections plotted in *ESRI* Arc View 3.3 GIS (left) and processed cross-sections following initial removal of ‘problem’ data and application of automated routines (right).

Data Projection

The 1992-1999 (inclusive) files were obtained in the North American Datum 1927 (NAD27) Mississippi West State Plane coordinate system. The 2001-2001 (inclusive) files were obtained in the North American Datum 1983 (NAD83) Mississippi West State Plane coordinate system.

The Corpscon (Corps Conversion) software, developed by the US Army Topographic Engineering Center (TEC), was used to convert all cross-section point coordinates to the regional standard, NAD 1983, Universal Trans Mercator (UTM) Zone 15 system. All elevations are given in feet and are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Cross-Section Screening

An initial task was to use the LWRP information supplied by the Vicksburg District to 'screen' the cross-sections and determine which ones extended to elevations corresponding to LWRP, +5, +10, +15, +20, +25 and +30 ft above LWRP. This was essential to check the feasibility of analyzing channel geometry distributions for all years between 1992 and 2001 (with the exception of 1998 for which no survey is available) and elevations proposed. The results are listed in Table 1.

year	<LWRP	>LWRP	>LWRP +5ft	>LWRP +10ft	>LWRP +15ft	>LWRP +20ft	>LWRP +25ft	>LWRP +30ft	total
1992	188	68	51	34	19	10	0	0	256
1993	196	60	25	8	0	0	0	0	256
1994	126	153	119	62	6	1	0	0	279
1995	196	60	25	6	0	0	0	0	256
1996	36	220	185	5	5	0	0	0	256
1997	23	233	230	226	213	194	81	6	256
1999	127	132	79	48	5	0	0	0	259
2000	245	11	0	0	0	0	0	0	256
2001	50	205	184	163	131	100	87	64	255

Table 1. Initial screening of cross-sections

The results indicate that the extent of cross-sections varies between years. For example, that for 2000 is of very limited use as this was a very low flow year. In performing the analysis, emphasis will inevitably be placed on years with a more complete range of elevations such as 1992, 1994, 1997 and 2001.

The data in Table 1 take a strict approach whereby a cross-section is excluded if either of the end points lies below the specified level, even by a small amount. However, there are many cross-sections which just fall short of this screening criterion. A sensitivity analysis was performed to investigate whether limited extrapolation of cross-sections would improve matters significantly. The final scheme adopted was to allow extrapolation of the bank for a vertical distance of 10 feet if the bank slope (vertical to horizontal) is at least 0.2 (which accounts for where the low bank is bounded by a revetment and the investigators have some confidence that the slope of the section is likely to continue to a higher elevation) and extrapolation for a vertical distance of 5 feet if the bank slope is between 0.025 and 0.2. Cross sections with smaller side slopes were left unchanged. The revised cross-section screening results are listed in Table 2 and the number of cross

sections extending above the LWRP (both end points) that have been added to the list is illustrated in Figure 4.

year	<LWRP	>LWRP	>LWRP +5ft	>LWRP +10ft	>LWRP +15ft	>LWRP +20ft	>LWRP +25ft	>LWRP +30ft	total
1992	159	97	67	50	31	17	7	0	256
1993	158	98	63	27	6	0	0	0	256
1994	107	172	153	119	66	14	1	1	279
1995	141	115	77	35	5	0	0	0	256
1996	30	226	201	175	36	5	4	0	256
1997	14	242	233	230	224	203	190	115	256
1999	60	199	158	82	48	11	1	0	259
2000	126	130	35	0	0	0	0	0	256
2001	41	214	208	187	160	134	105	91	255

Table 2. Revised screening of cross-sections, following limited extrapolation.

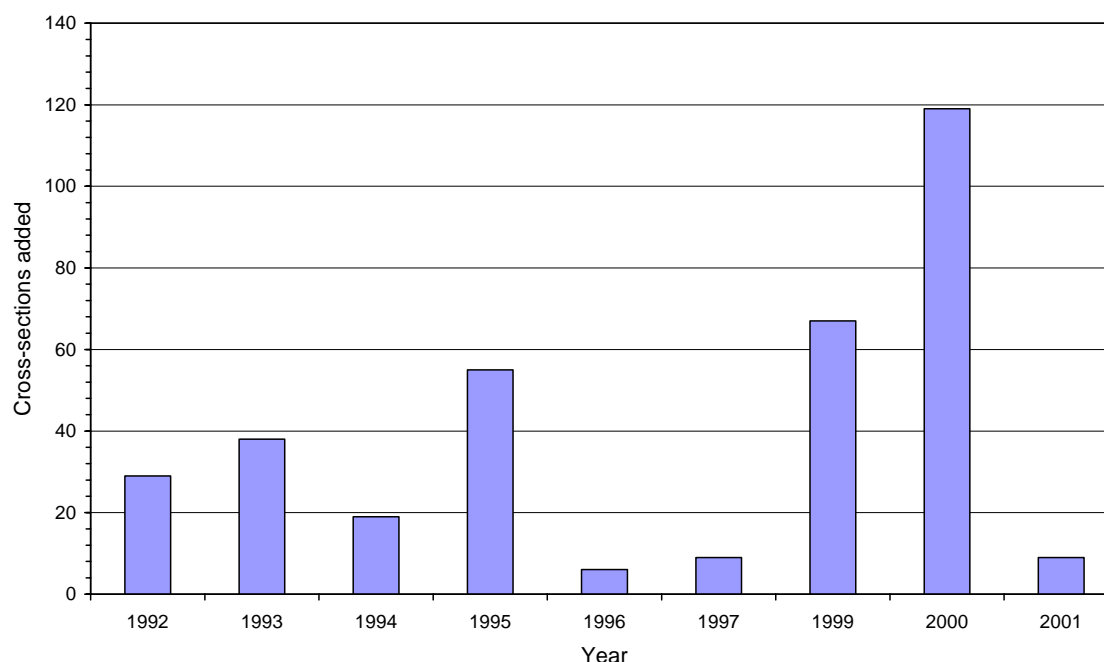


Figure 4. Additional cross-sections extending above the LWRP following limited extrapolation.

The effect of the extrapolation process is varied between years, but overall has assisted in developing a substantially larger dataset for undertaking channel geometry analysis. At the LWRP elevation, the extrapolation has had a negligible impact in 1996, 1997 and 2001 (the latter two years being high water years) and a large impact in 1995, 1999 and 2000. However, at higher elevations, there has been a large impact in the higher water years, with, for example, an additional 170 cross-sections added in 1996 for 10 feet above

LWRP and an additional 109 cross-sections added in 1997 for both 25 feet and 30 feet above LWRP.

Bends and Crossings

It was considered at the outset that the channel geometry of crossings may differ significantly from that of pools in meander bends. Thus, a channel planform curvature parameter has been used to discriminate between bends and crossings, with separate analyses performed on each. The technique assumes that meander bend and crossing 'reaches' can be suitably distinguished based solely on changes in channel curvature between successive cross-sections. The process adopted involved: i) plotting the point in each cross-section with the lowest elevation, the thalweg (not the centerline as there are marked discontinuities along the study reach where the length of cross-section changes markedly over a very short distance); ii) smoothing the line using MapInfo GIS software (particularly within the crossing reaches where the deepest point in the channel is not always clearly defined); applying a macro to calculate the absolute (positive) change in channel direction between cross-sections, and; iv) applying a smoothing algorithm using Tecplot post-processing and visualization software to define similar reaches.

The approach revealed that bend and crossing reaches in the study reach tend to be distinguished by the threshold of about 5 degrees of change in channel direction. The final classification scheme adopted is as follows: pool reaches are defined as having a change in direction of at least 6 degrees; crossing reaches are defined as having a change in direction of less than 4 degrees; transitional reaches are defined as having a change in direction of between 4 and 6 degrees. These reaches are illustrated in Figure 5.

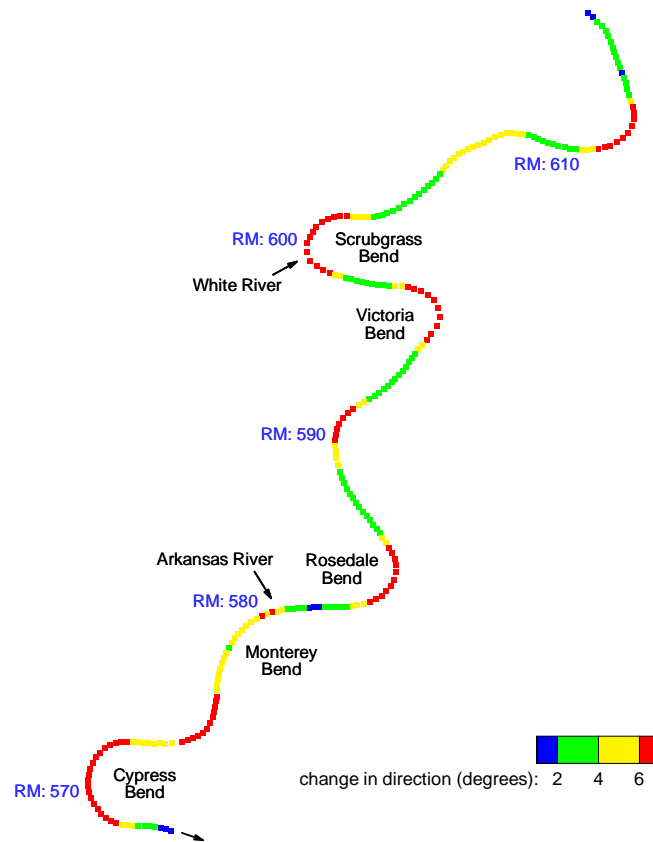


Figure 5. Change in channel direction (positive) between cross-sections.

Divided Reaches

Although most cross-sections are comprised of a single channel, it was necessary to investigate whether the variability of channel geometry in this reach of the Lower Mississippi is influenced by the presence of divided reaches with secondary channels. These are often found in asymmetrical cross-sections with wide point bars and have particular ecological importance.

To investigate this, where secondary channels are present in a cross-section, the width, average depth, etc., were calculated for both the entire cross-section (all channels added together) at the specified elevation and then for just the primary channel, defined as that part of the cross-section containing the lowest elevation. The effect of this, though, on the channel geometry analysis is shown to be only very minor when considering the reach as a whole and only very localized when comparing the geometry of individual cross-sections, simply because secondary channels are not common features and where they exist they tend to become included within the primary channel either at elevations below the LWRP or just higher.

STUDY APPROACH AND METHODOLOGY

Scientific Basis for At-a-station Channel Geometry Analysis

Hydraulic Geometry Analyses

At-a-station hydraulic geometry deals with discharge-related variations in cross-sectional and hydraulic parameters at a particular location in the river. Downstream hydraulic geometry deals with spatial changes in cross-sectional and hydraulic parameters at a single, reference discharge (usually bankfull discharge). Relations for width, average depth, maximum depth, and velocity have a similar form in both at-a-station and downstream hydraulic geometry, although the regression constants and exponents differ markedly. For example, the exponent for at-a-station width variation is usually lower than that for downstream change.

The variation of cross-sectional parameters with discharge at a section occurs for three main reasons. At very low discharges, boundary shears stress is below the threshold value for mobilization of the channel bed and bank materials. Hence, the boundaries of the channel are effectively fixed and the variation of geometry reflects only the rise of water level as discharge increases and resistance decreases (mostly as a function of decreasing relative roughness). This is the first stage of hydraulic geometry analysis. When the bed material (and perhaps the bank material) becomes mobilized, changes in channel geometry reflect not only increasing discharge and changing flow resistance, but also deformation of the cross-section through scour and fill. This is the second stage. When bankfull stage is reached, flows begin to spill onto the floodplain – complicating the geometry and indeed the practical definition of the ‘channel’. This is the third stage of hydraulic geometry.

The variation in cross-sectional parameters with bankfull discharge reflects the control exerted on alluvial channels by this ‘dominant’ or ‘channel forming flow’.

Numerous downstream and at-a-station analyses have been carried out. Coherent sets of equations have been derived for the downstream hydraulic geometry of gravel and sand-bed rivers, but no overall pattern has emerged (Knighton, 1998).

In this project, it was proposed to perform hydraulic geometry analyses for a selected sub-reach of the Lower Mississippi River. However, it became apparent early in the study that this was neither possible nor desirable.

Examination of discharge records for the selected study reach revealed that there was no marked change in discharge along its length. This is the case because, on the scale of the Lower Mississippi, the reach is relatively short and because the discharges added by the White and Arkansas Rivers are much smaller than that in the trunk stream. Hence, downstream hydraulic geometry analysis is meaningless as there would be no variation in

the reference flow. Also, very few of the available cross-sections extend to anything near the bankfull stage. This would rule out use of bankfull discharge as the reference flow for downstream hydraulic geometry analysis, greatly limiting the validity of the approach. For these reasons, it was agreed by the contractor and USACE staff that downstream hydraulic geometry was not feasible for the selected sub-reach.

With respect to at-a-station hydraulic geometry analysis, a problem occurs as there is only one gauging station in the vicinity of the study reach. It is difficult to determine the discharge that corresponds to any particular stage at cross-sections remote from a gauging station. Reliance must be made on estimation of local flow resistance and the effects of non-uniform, gradually varied flow. This degrades the quality of the analysis as the detail of changes in flow resistance with stage is one of sources of uncertainty that hydraulic geometry analysis is intended to address. In any case, discussion with USACE staff in Vicksburg confirmed that their primary interest concerns stage-related (rather than discharge-related) changes in cross-sectional parameters. Indeed, this was implicit in the original proposal which specifies in the *scope of work* that:

‘Hydraulic geometry would include width, depth, hydraulic radius, wetted perimeter, cross-sectional area and other parameters (details to be agreed with ERDC in the project formulation phase) for the river at LWRP and heights at +5 feet increments, up to a maximum elevation of +30 feet.’

To clarify the difference between what was required by USACE and conventional at-a-station hydraulic geometry, it was agreed by all parties that the project would perform at-a-station *channel* geometry analysis rather than *hydraulic* geometry analysis. That is, all analyses would be conducted by reference to a specified stage above LWRP, rather than to a specified discharge.

Channel Geometry Analysis

For each cross section, the width, average depth, maximum depth, wetted perimeter, hydraulic radius and cross-section area have been calculated. This involved two analytical stages:

Initially a macro was used to prepare the data into a suitable format for calculating the above parameters. This macro included: i) extending the cross-section according to the criteria previously discussed; ii) calculation of the distance between cross-section points; iii) calculating the elevation difference between the interpolated LWRP and both cross-section end points and then, recording the lowest difference (rounded to whole feet). This became the largest ‘Elevation above LWRP’ for which the channel geometry analysis was undertaken (and also facilitated the development of Tables 1 and 2); iv) calculation of the Easting and Northing coordinates of the deepest point in each cross-section, (used to calculate changes in channel direction for defining the bend and crossing reaches).

The second stage involved the actual calculation of the channel geometry parameters and utilized a macro to undertake the calculations, firstly for each cross-section in each

annual survey at the LWRP elevation, and then repeatedly for one foot increments above the LWRP to a maximum of 30 feet, or once the elevation is greater than both cross-section end points. The procedure involved dividing each cross-section into vertical panels, calculating each panel's top width, wetted perimeter (for channel boundaries only) and cross-section area and then summing these values to find their cross-section totals. The average depth is defined as the ratio of the cross-sectional area to the total width and the hydraulic radius is the ratio of the total cross-section area to the total wetted perimeter. The procedure was repeated for the main channel only in cross-sections where secondary channels exit (see discussion above).

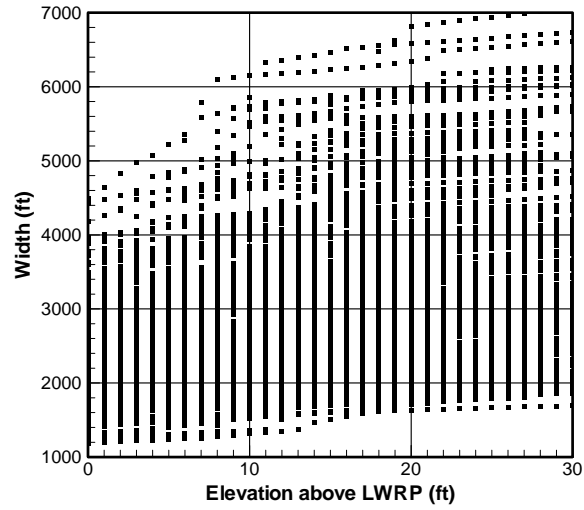
Probability Analysis

Conventional downstream hydraulic geometry analysis involves developing power-law relations to predict various channel geometry parameters. However, examination of the variation of width, average depth, etc., with elevation above LWRP did not reveal the type of close association that is normally required to define a best-fit regression line but instead exhibited wide ranging distributions. Fitting any type of best-fit line would misleadingly ignore this variability in channel geometry that is characteristic of the Lower Mississippi River. Thus, an alternative technique was required to 'capture' this variability and describe how it changes both with elevation above LWRP and between annual surveys, without any loss in information. The approach adopted is to assign each cross-section with a cumulative probability (from 0 to 1) for each channel geometry parameter, as illustrated for width in Figure 6.

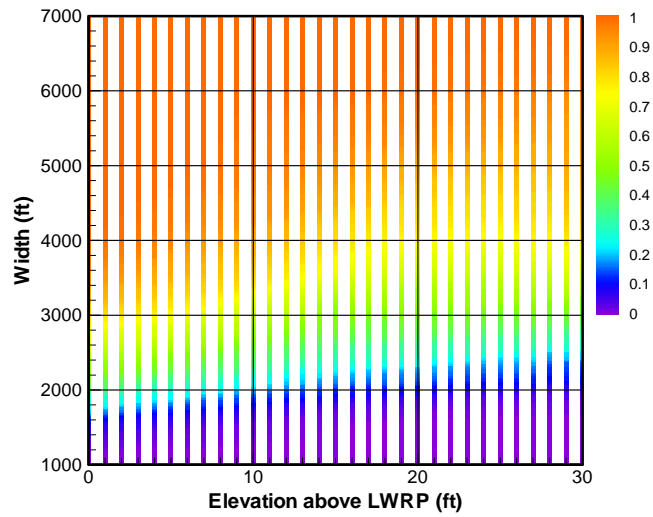
Initially, all values were plotted against increasing elevation above LWRP (Stage 1 in Figure 5 demonstrates the high degree of variability in width). For each elevation, cumulative probabilities were then assigned to each width value. These probabilities were then rectified to a regular grid by linear interpolation, firstly for each one foot increment above LWRP (Stage 2) and then across the remainder of the grid (Stage 3). Notably, the cumulative probabilities were only calculated if there were at least 10 cross-sections that extend above the specific elevation. Thus, many of the final plots do not extend as far as 30 feet above LWRP. Finally, the 50th percentile line was calculated and displayed in the final plots. A series of macros and the Tecplot visualization software were used to undertake these processes.

The procedure was undertaken for all cross-sections and then bends and crossings separately.

Stage 1:



Stage 2:



Stage 3:

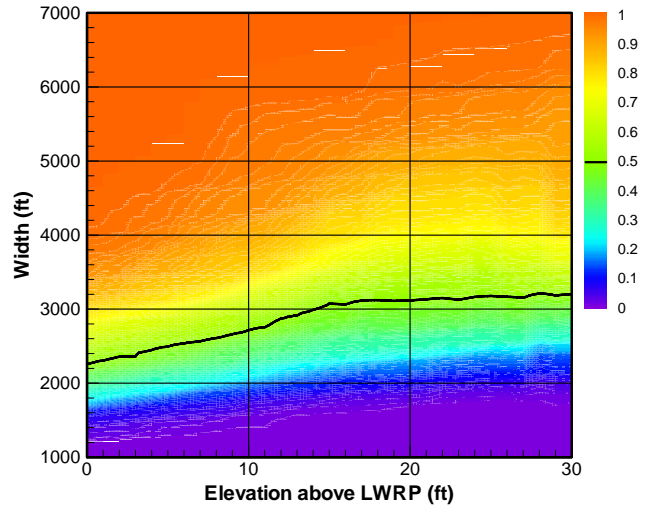


Figure 6. Development of cumulative frequency plots (the example shown is for width in the 1997 survey).

Spatial Analysis

A spatial analysis was undertaken to explore how channel geometry varies within the study reach itself and between years. The approach involved plotting the location of each cross-section as a series of symbols along the smoothed thalweg and shading according to the value of width, average depth, etc. In addition to the basic geometry parameters, values of channel conveyance, defined as the product of area and hydraulic radius to the power of two thirds, were also calculated.

Although between 1992 and 2001, the hydrographic surveys were repeated at approximately the same location, for visualization purposes (and to facilitate the temporal analysis, described below) the 2001 survey was used to locate each cross-section for all years. Values for cross-sections that could not be calculated (i.e. end points do not extend above the specified elevation) were interpolated from neighboring sections. If less than 10 sections were available, then no values are given in the final plots as the specified elevation above LWRP is considered to be unsuitable for the spatial analysis.

Temporal Analysis

The temporal analysis uses the same graphical format as the spatial analysis but the cross-section symbols are shaded according to the change in average depth and change in cross-section area between annual surveys and over the period 1992-2001. Over the full time period, changes in average depth are indicative of aggradation/degradation within the reach and changes in cross-section area highlight zones of scour and fill. The temporal analysis has only been undertaken for the LWRP elevation as it was judged that the degree of interpolated values at higher elevations could potentially generate misleading patterns of channel change.

RESULTS

At-a-Station Channel Geometry

Interactive CD-ROM

Overview

All results are stored as picture files on the CD-ROM that accompanies this report. In total there are 1,305 plots (324 for the probability analysis; 945 for the spatial analysis and; 36 for the temporal analysis). The CD-ROM is interactive and permits the user to navigate through the plots and scroll through different years to examine changes in channel geometry.

The CD-ROM contains all the cross-section data with Eastings and Northings referenced to the NAD 1983, Universal Trans Mercator (UTM) Zone 15 coordinate system. All elevations are given in feet and are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). The data are stored as text files under the sub-directory: 'NAD83 UTM Zone 15 sections'.

The main functionality of the CD-ROM is the interactive browser facility. Instructions for using the browser are as follows:

Running the CD-ROM

The CD-ROM should start automatically. Alternatively, the browser can be started by running the program 'autorun.exe'. The user is presented with a menu with four options (Figure 7). The first option is to 'Load VB runtime files'. This installs the necessary Visual Basic 6 library files into the Windows directory of the user's PC and is recommended to ensure the browser program functions correctly (this only needs to be done once). The second option is to 'Run interactive CD' and will initiate the browser program. The third option is 'About this CD' and provides information about the project, contract, investigators, etc. The CD should be referenced using the project information provided here. Finally, the fourth option allows the user to 'Exit' the software.

Running the Browser Program

It is recommended that the browser is run at a screen resolution of at least 1024*768. The browser presents the user with a choice of three graphical interfaces, selected using the option boxes at the top of the screen: i) Cumulative Frequency Analysis (Figure 8); ii) Spatial Analysis (Figures 9, 10), and iii) Temporal Analysis (Figure 11). The graphs can be copied to the windows clipboard using the button at the bottom of the screen. In all cases, values which lie outside the range shown in the legend assume the colour of the legend's end points.

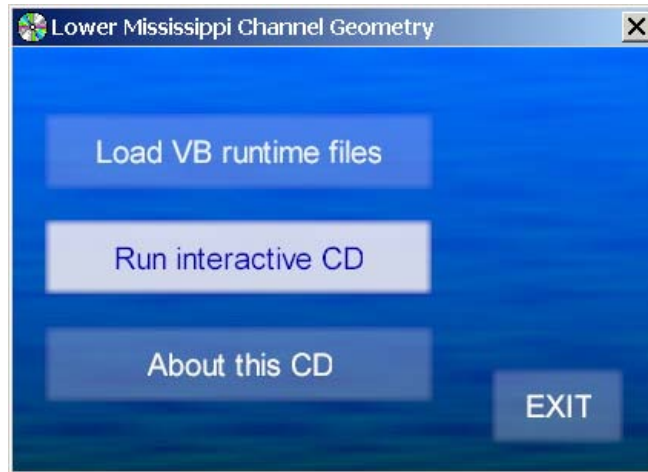


Figure 7. Interactive CD-ROM main menu

The cumulative frequency (or probability) analysis screen presents a standard graph showing how the cumulative distribution of the selected channel geometry parameter varies with elevation above LWRP. The graph automatically updates following changes made by the user in three list boxes giving choices of year, geometry parameter and reach type (all, bends or crossings). The graph can also be updated for channel geometry in the main portion of the channel only (removing secondary channels) using the check box provided. The graph is blanked out for all elevations for which there are only 10 or less cross-sections available to derive a cumulative probability distribution.

The spatial analysis screen displays the study reach as a series of color-coded symbols which relate to the magnitude of the chosen channel geometry parameter for each cross-section. The graph can also be updated for changes in year and elevation above LWRP (5 feet increments up to 30 feet). The study reach can be zoomed in (and out) by clicking on the magnifying glass. Once zoomed in, the mouse can be used to scroll within the study reach. It is also possible to highlight in white all cross-sections which have interpolated values by clicking on a check box near the bottom of the window (see Figure 9).

The temporal analysis screen uses a similar interface to that of the spatial analysis. The user can select different periods (between years and for the period 1992 to 2001) to investigate changes in either average depth or cross-section area at the LWRP elevation using the list boxes provided. The zoom facility is also available.

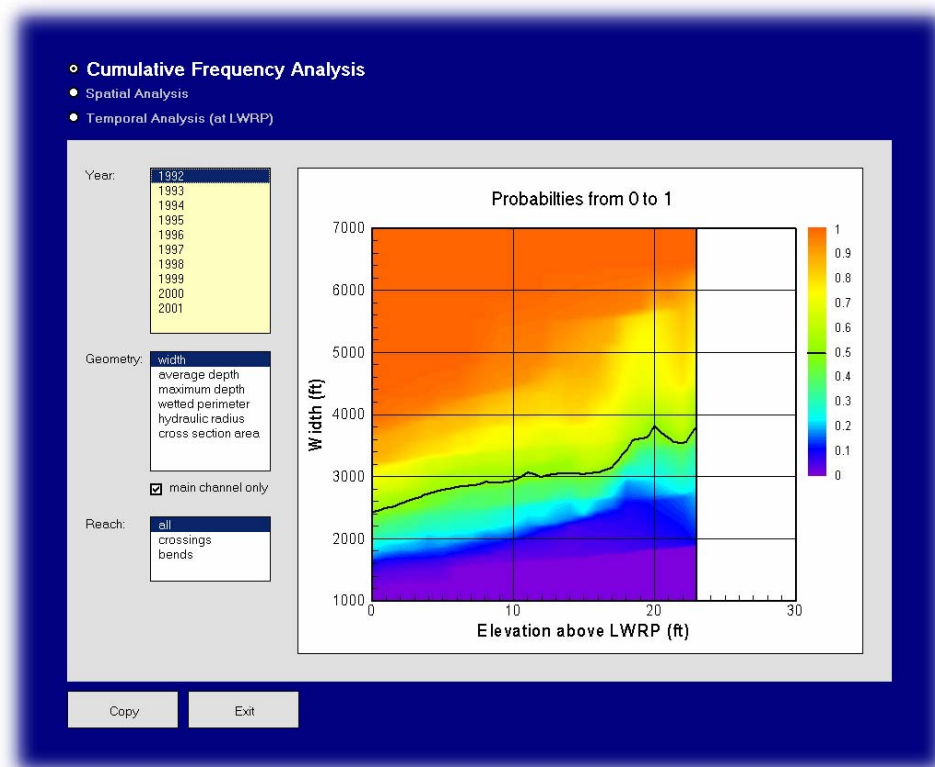


Figure 8. Browser Program: Cumulative Frequency (Probability) Analysis Window.

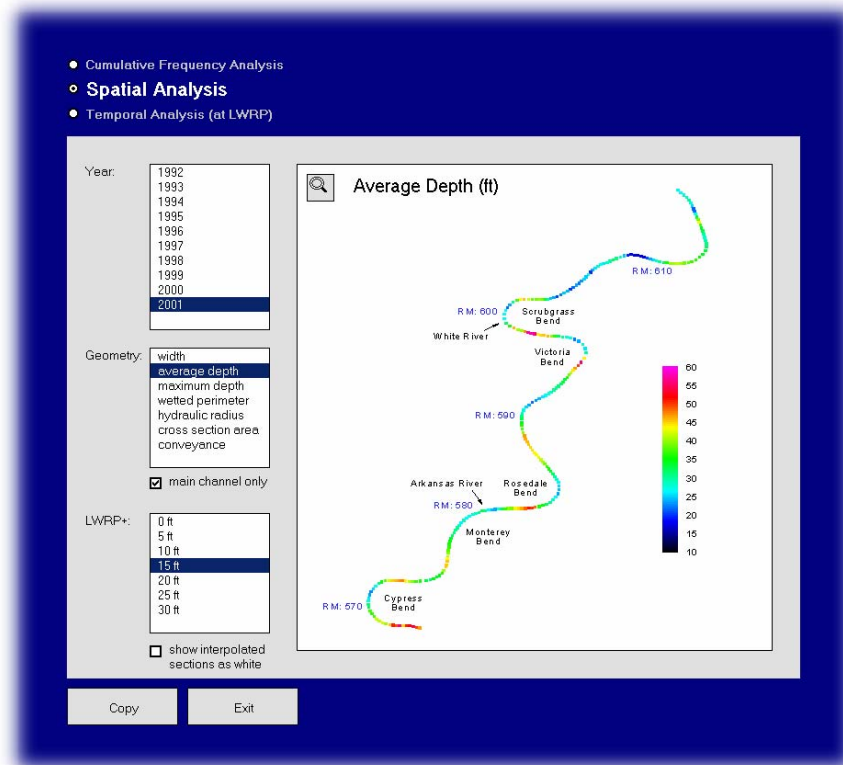


Figure 9. Browser Program: Spatial Analysis Window.

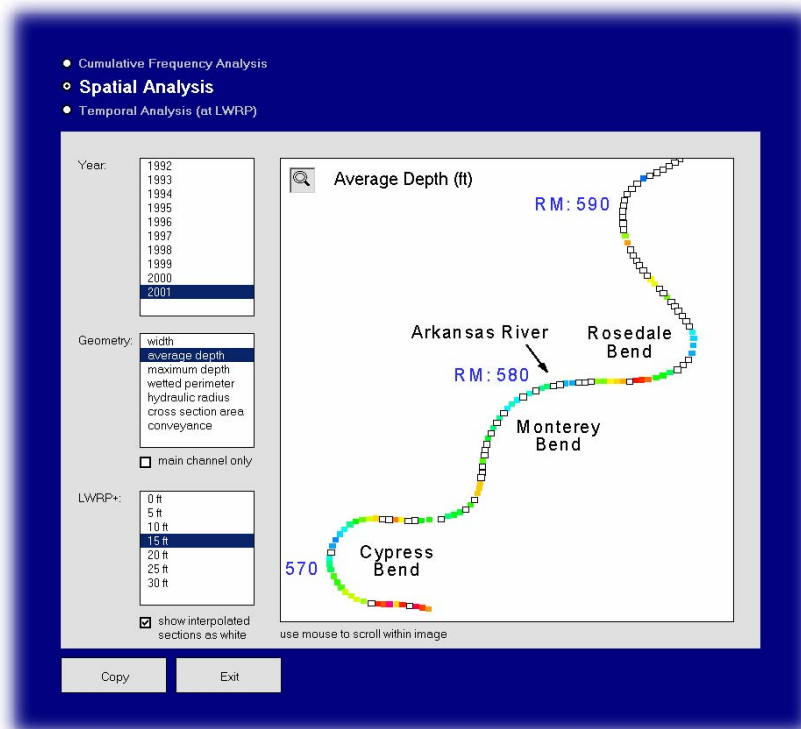


Figure 10. Browser Program: Using the Zoom Facility and Showing Interpolated Values in the Spatial Analysis Window.

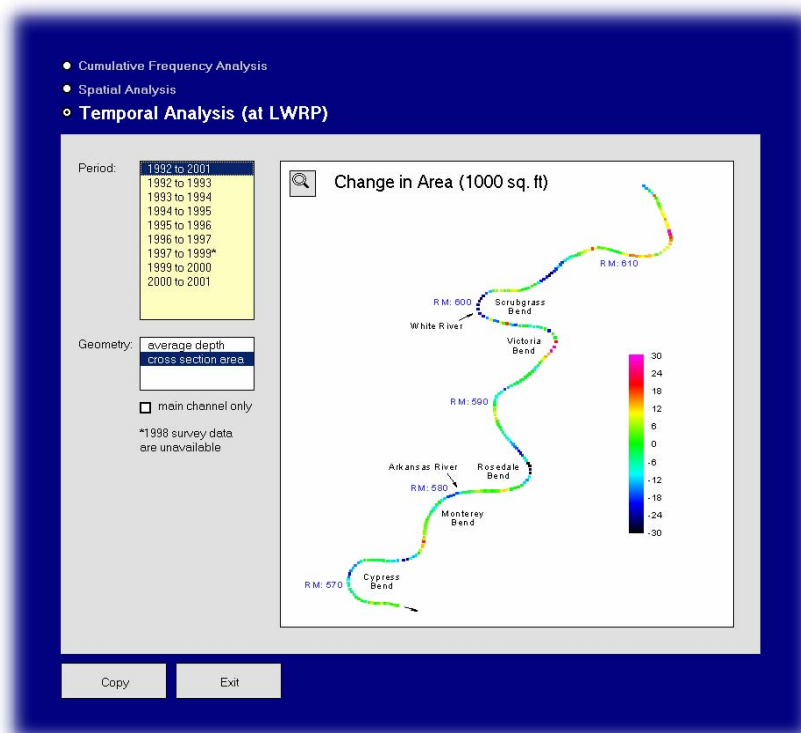


Figure 11. Browser Program: Temporal Analysis Window.

Example Results for At-a-Station Channel Geometry

Examples of the results from the probability analysis to better understand at-a-station channel geometry are illustrated in Figure 12. The full set of results is provided on the interactive CD-ROM.

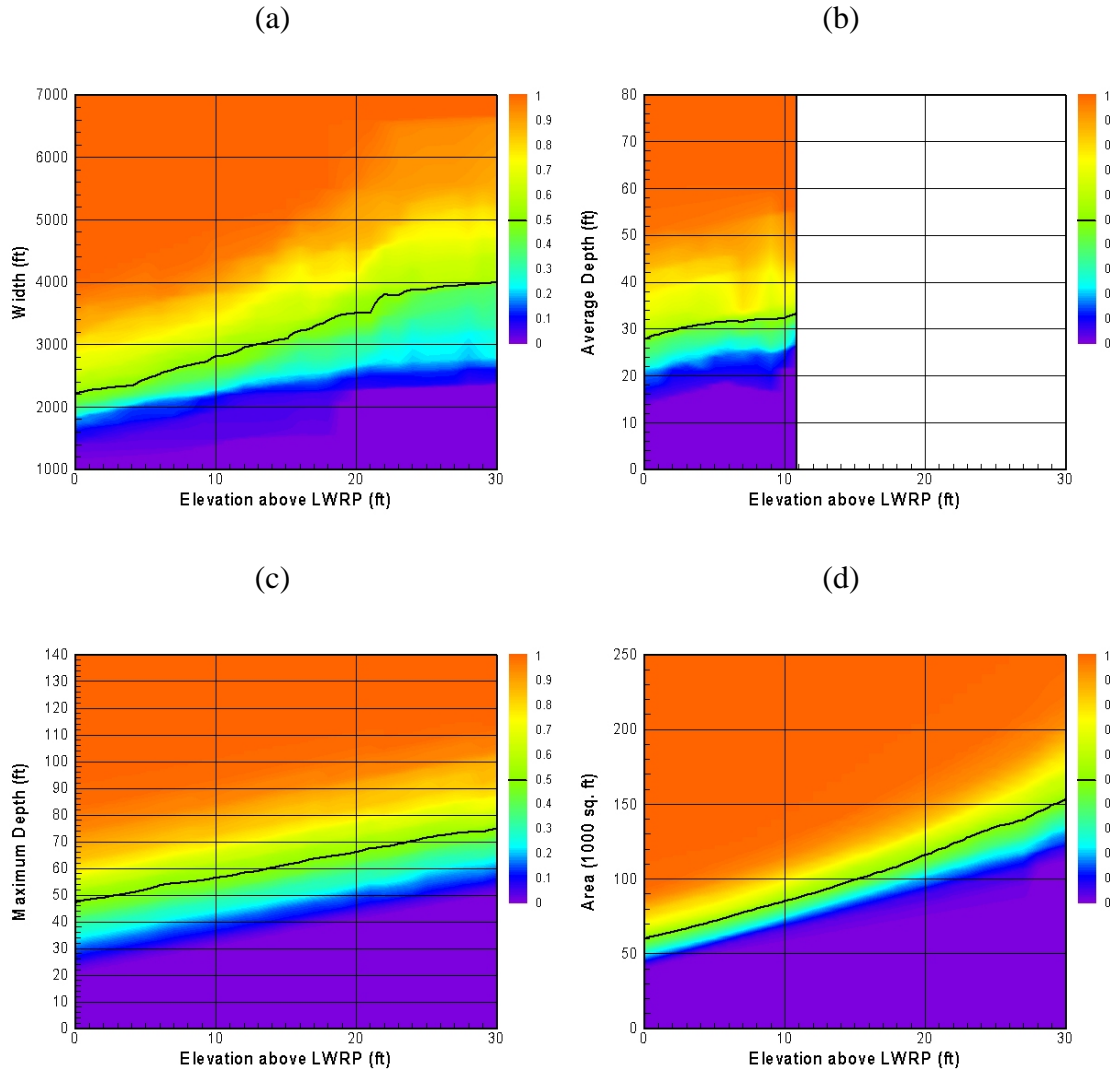


Figure 12. Example Results from the Probability Analysis: (a) width in 2001 for bends; (b) average depth in 1995 for crossings; (c) maximum depth for all sections in 2001; (d) area in 1997 for all sections.

Spatial Variability and Adjustments

Examples of the results from the spatial analysis to examine how channel geometry parameters vary along the study reach are illustrated in Figure 13. The full set of results is provided on the interactive CD-ROM.

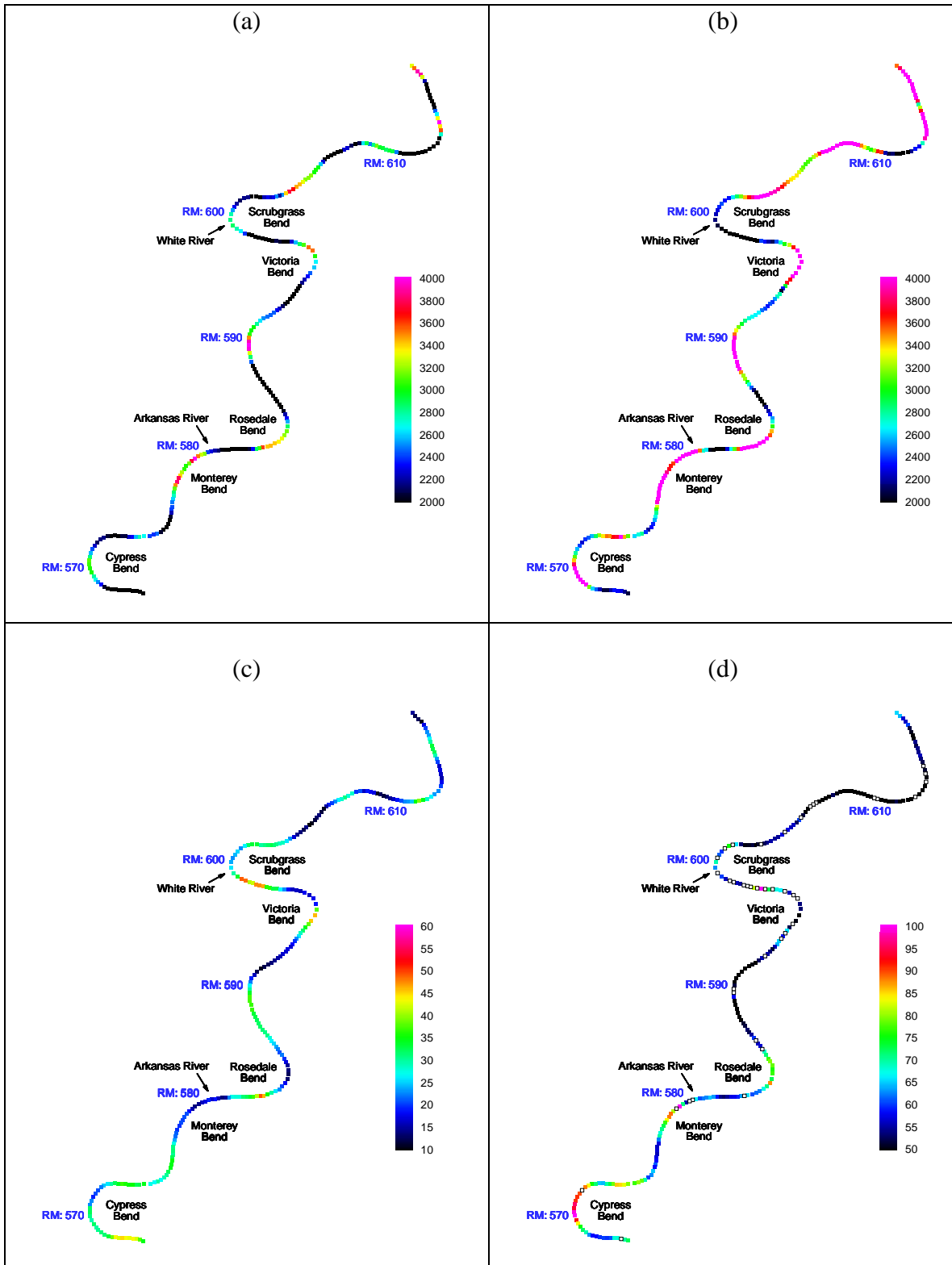


Figure 13. Example Results from the Spatial Analysis: (a) width (ft) in 1997 at LWRP; (b) width (ft) in 1997 at 20 feet above LWRP; (c) average depth (ft) in 2001 at LWRP; (d) area (1000 sq. ft) in 1996 at LWRP (interpolated values shown in white).

Temporal Variability and Adjustments

Examples of the results from the temporal analysis to examine how average depth and cross-section area within the study reach change between years and over the full period 1992-2001 are illustrated in Figures 14 and 15. The full set of results is provided on the interactive CD-ROM.

Aggradation/Degradation

Aggradation and degradation can be inferred from the results for average depth as depth is referenced to LWRP – which constitutes a fixed reference elevation that does not change through time.

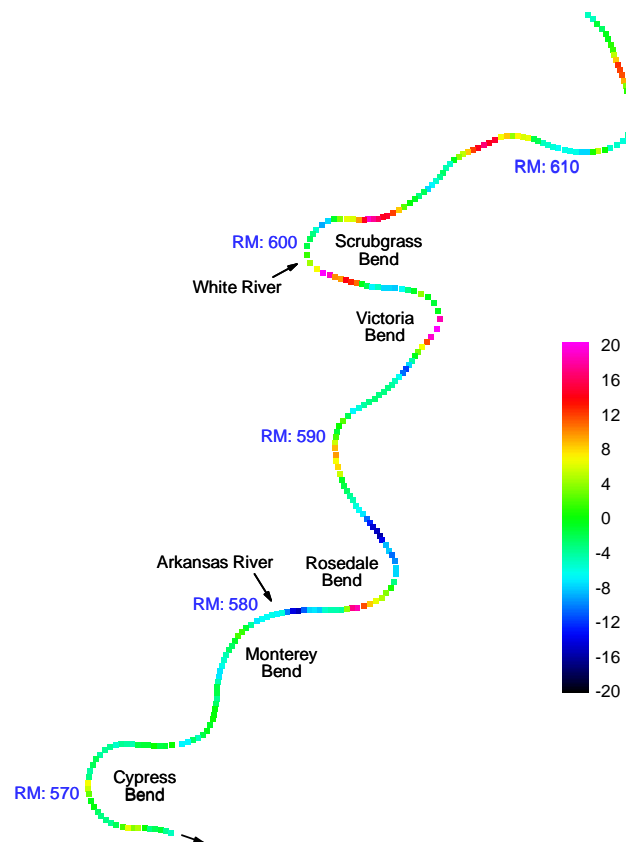


Figure 14. Changes in average depth (ft) between 1992 and 2001, indicative of aggradation and degradation.

Scour/Fill

Scour and fill can be inferred from changes in the cross-sectional area, as this is also referenced to LWRP, which provides a time-invariant reference plane.

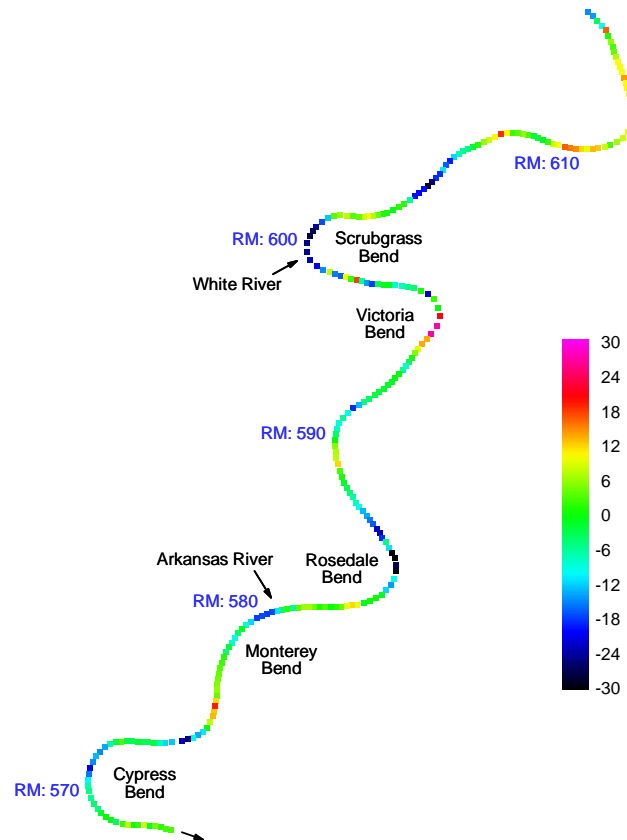


Figure 15. Changes in area (1000 sq. ft) between 1992 and 2001, indicative of scour and fill

INTERPRETATION AND COMMENTARY

Indicated Adjustments 1992 to 2001

Channel Geometry Analysis

Width

Channel Width at LWRP fluctuates about 2,200 feet (+/- 200 ft). Bends are generally wider than crossings.

Crossing width at LWRP fluctuates about 1,900 feet. In 2000 1,750 but in 2001 rose to 2,200.

Bend width at LWRP fluctuates between 2,700 – 2,050. High 1992 low 2000 – may be discharge related.

High flow channel width (based on +20) is consistent in 1997 and 2001 at around 3,100ft.

Slope of median line for channel width is consistent between lwrp and +10 for years with sufficient data.

Slope of median line for crossing width is lower than that for bends between lwrp and +10 for years with sufficient data.

Average depth

Channel average depth at LWRP fluctuates about 28 feet (ranging between 25 and 32ft). Bend and crossing average depths are generally similar. Differences (either way) seem to be related to discharge. In high water years depths converge to be approximately equal in bends and crossings.

Crossing average depth at LWRP fluctuates about 28 feet. Bend average depth at LWRP is more variable and fluctuates 29ft. Difference may be explained by different cross-section shape at bends, which tend to be less uniform and rectangular and more variable in point depth across the wetted section.

High flow channel average depth (based on +20) is around 33 to 37ft.

Slope of median line for channel average depth is consistent between LWRP and +10 for years with sufficient data.

Inter-quartile range for crossing average depth is wider than that for bends between for years with sufficient data.

Maximum depth

Note: relationship with elevation above LWRP should be linear by definition. Deviations from linearity reflect data limitations. This occurs because as stage increases, the number of cross-sections with data decreases, producing changes in the sampled data and non-linearity in the median line.

Channel maximum depth at LWRP fluctuates about 52 feet (ranging between 46 and 54ft). Bend and crossing maximum depths are generally similar. Differences (either way) seem to be related to discharge. In high water years depths converge to be approximately equal in bends and crossings.

Crossing maximum depth at LWRP fluctuates about 52 feet. There is an anomalously low value (41 ft) in 2001. Bend maximum depth at LWRP is more variable and fluctuates 50ft. Lack of difference may be explained by the fact that what is plotted is the median maximum depth for several cross-sections and the fact that the deepest point in a bend pool may actually be found downstream of the planimetric bend, in the crossing reach downstream.

High flow channel-averaged maximum depth (based on +20) is around 66 to 72ft.

Slope of median line for channel maximum depth is consistent between LWRP and +10 for years with sufficient data.

Inter-quartile range for crossing average depth is considerable wider than that for bends between for years with sufficient data.

Wetted perimeter and hydraulic radius

Wetted perimeter and hydraulic radius are indistinguishable from width and average depth respectively due to high width depth ratio of channel.

Cross-sectional area

Channel cross-sectional area at LWRP fluctuates about 60 thousand square feet (ranging between 55 and 65 thousand ft²). Bends tend to have larger areas than crossings.

Crossing area at LWRP fluctuates about 55 thousand square feet. Bend average depth at LWRP is more variable and fluctuates 65 thousand square ft. Difference may be explained by different cross-section shape at bends, which tend to be less uniform and rectangular and more variable in point depth across the wetted section.

High flow channel area (based on +20) is around 110 to 120 thousand ft². Crossings 110 to 120, Bends 120 to 125.

Slope of median line for channel cross-sectional area is consistent between LWRP and +10 for years with sufficient data.

Inter-quartile ranges for cross-sectional area at both bends and crossings are narrow.

Spatial Analysis

Width varies between bends and crossings, but there are no marked reach-scale differences. There may be a tendency for the LWRP channel to widen between 1992 and 1997, and to be narrower in 2001 (especially Scrubgrass Bend).

Average depth at LWRP displays no reach-scale trend. Greatest average depths are found in cross-sections at bend exits or in the upstream part of crossing reaches.

Deepest cross-sections are at bend exits or upstream of inflection point in crossing reaches. No reach-scale spatial variation can be discerned.

Cross-sectional area at (LWRP and +5) appears to be larger in and downstream of Rosedale Bend (in 1997 and 2001).

Temporal Analysis

At the reach-scale, average depth changes little between 1992 and 2001. About seventy five percent of changes are less than +/- 5 ft in magnitude. Changes that are apparent indicate a tendency for bed scour at and downstream of bend exits (for example average depth increases in excess of 10 ft at Rosedale, Monterey and Cypress bends in 1993-4) and aggradation on the crossings downstream.

Examination of changes on a year to year basis reveals short-term adjustments that are not sustained over the longer period. Most activity occurs in 1993-1994 and 2000-2001, possibly related to the higher flow years. There was over 20 ft of bed scour at Victoria Bend between 1997 and 1999, which may be related to local adjustment or could be a response to bendway weirs installed at that bend in 1995, which did not initially operate as intended.

In 2000-1 there appears to be local sedimentation in bend entrances and bed scour at bend exits.

The absence of reach-scale changes in average depth indicates a lack of aggradation or degradation that is consistent with the reach being dynamically stable (as indicated by specific gauge analyses performed by Biedenharn et al.).

Examination of cross-sectional areas reveals that between 1992 and 2001 there have been notable reductions in area around the Scrubgrass and Rosedale Bends. At both bends, area has decreased by more than 25,000 square feet. Crossings have generally changed by less than +/- 8,000 square feet between 1992 and 2001.

In terms of year on year changes, there is little activity in 1992-93, but in 1993-94 there was more scour and fill. This did not, however, follow any systematic pattern related to bends and crossings. Scour and fill in 1994-5 tended to reverse the changes that occurred in the previous period. There was little activity in 1995-6 , 1996-7 and 1997-99. Apparent activity at Victoria Bend in 1999-2000 and at other location in 2000-1 is probably due to lack of data for 2000.

RECOMMENDATIONS FOR FURTHER RESEARCH

The channel geometry analysis performed in this project has proven the viability of extracting useful morphological information from hydrographic survey records. The results demonstrate that the cross-sectional geometry of the Lower Mississippi River is highly variable in both time and space. Despite this, the fact that median values of key at-a-section parameters such as width, average depth, maximum depth, cross-sectional area and conveyance ($AR^{2/3}$) change in an orderly manner with respect to stage change gives an insight into the stable morphology of the river.

1. *On this basis it is recommended that further analysis of the existing channel geometry database be pursued to extract median values for use as design parameters when re-sectioning or training reaches to improve flood control or aid navigation.*

Temporal changes in cross-sectional geometry demonstrate that the equilibrium of this reach is highly dynamic. Year-on-year changes observed in the study reach are larger than the net change observed over the decadal period of record, and there is some evidence that morphology responds to flood magnitude. However, even a decade long period of record is short for a river as large as the Lower Mississippi and no firm conclusion concerning the adjustment status of this reach can be drawn from the analysis presented herein. For example, Biedenharn and Watson's specific gauge analysis suggests that this reach may now be stable as the wave of incision generated by the cutoffs has migrated upstream. This cannot be verified at present as the period of record does not commence until 1992, by which time incision in this sub-reach would have ended. Establishing the historical and current status of morphological channel adjustment would be very useful in decision making concerning engineering measures to deal with sediment and morphology-related problems in this sub-reach.

2. *It is recommended that the period covered by channel geometry analysis is extended back to the end of the cutoff period using additional annual hydrographic survey records available at the Vicksburg District USACE.*

The sub-reach covered in this short project is only a fragment of the Lower Mississippi River. It is located within the 'hinge zone' proposed by Biedenharn and Watson. While the results of this project support and are consistent with their interpretation of specific gauge records, it would be very informative to apply the channel geometry analysis technique to other selected reaches in zones believed to be either incising or aggrading.

3. *It is recommended that two additional sub-reaches be investigated using the channel geometry analysis technique. Sub-reaches should be selected to be representative of the degrading and aggrading zones in Figure 1.*

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CHAPTER 3 GENERAL DESIGN CONSIDERATIONS

General design considerations for grade control can become as involved in structural and geotechnical engineering as a major concrete arch or earth-filled dam; however, the intent of this discourse is to limit the design discussion to those topics primarily considered by hydraulic engineers. General literature devoted to hydraulic structures (Novak *et al.* 1997; USBR 1987; U.S. Army Corps of Engineers (USACE) 1991) should be consulted for a detailed discussion of structural and geotechnical issues. Generalized guidelines to be considered by the hydraulic engineer should include the following topics:

- Develop the project goals and relate how grade control may be an element in the overall stream rehabilitation goal.
- Define the hydrology and hydraulics of the stream, including desired sediment yield, continuity of water and sediment, drainage and flood-control requirements, and habitat considerations.
- Consider alternatives to grade control, for example, if right-of-way is available, perhaps the construction of a meandering channel may provide a reasonable solution.
- Review the types of grade control structures that are commonly used, and make a preliminary selection of the type of structure required to satisfy constraints and design objectives.
- Conduct hydraulic and geomorphic analyses to determine stream and watershed stability, and select preliminary sites for construction.
- Conduct a field review of selected sites for local conditions. Local drainage, infrastructure, channel planform, highly erodible soils, or soils that require special considerations with respect to seepage, ecological or cultural assets, and other local factors may require special consideration or re-siting.

- Use specific design criteria for the specific structure selected and design the structure for each site.
- Conduct a final hydraulic and geomorphic analysis to assure that the final design satisfies channel stability and sediment yield requirements.
- Establish a post-construction monitoring and feedback plan to assure that the structures function as planned, and that lessons learned in the design and construction of each project can be documented for future use.

3.1 TYPES OF STRUCTURES

Selection of the type of structure to use is an important general decision in siting and spacing grade control structures. There are certain features that are common to most grade control structures. These include a control section for accomplishing the grade change, a section for energy dissipation, and protection of the upstream and downstream approaches. However, there is considerable variation in the design of these features. For example, a grade control structure may be constructed of riprap, concrete, sheet piling, treated lumber, soil cement, gabions, compacted earth fill, or other locally available material. Also, the shape (sloping, stepped, or vertical drop) and dimensions of the structure can vary significantly, as can the various appurtenances (baffle plates, end sills, etc.). The applicability of a particular type of structure to any given situation depends on a number of factors such as: hydrologic conditions, sediment size and loading, channel morphology, floodplain and valley characteristics, availability of construction materials, and project objectives, as well as the inevitable time and funding constraints. The successful use of a particular type of structure in one situation does not necessarily ensure that it will be effective in another. Some of the more common types of grade control structures used in a variety of situations are discussed in the following sections. Neilson *et al.* (1991) provide an international literature review on grade control structures with an annotated bibliography.

3.1.1 Loose-rock Structures

Perhaps the simplest form of a grade control structure consists of placing natural stone or other non-erodible elements across the channel to form a hard point. Some manufactured

concrete products may be used in place of stone. This type of structure encompasses rock sills, rock sills with impermeable cutoffs, artificial riffles, and sloping rock structures.

These types of structures are generally most effective for drop heights that are less than about 2 to 3 ft. In many applications, a series of loose-rock structures are placed relatively close together, effectively providing a greater drop height than a single structure. The series of loose-rock structures then provide a degree of conservatism in the design, as one element may reduce stress on the upstream element. Loss of one element may not mean loss of function for the total structure.

A series of rock sills, each creating a head loss of about 2 ft, was used successfully on the Gering Drain in Nebraska (Stufft 1965). The design concept presented by Whitaker and Jäggi (1986) for stabilizing the stream bed with a series of rock sills is shown in Figure 3.1. The sills in Figure 3.1 are bed control structures, which are simply acting as hard points to resist the erosion of the stream bed.

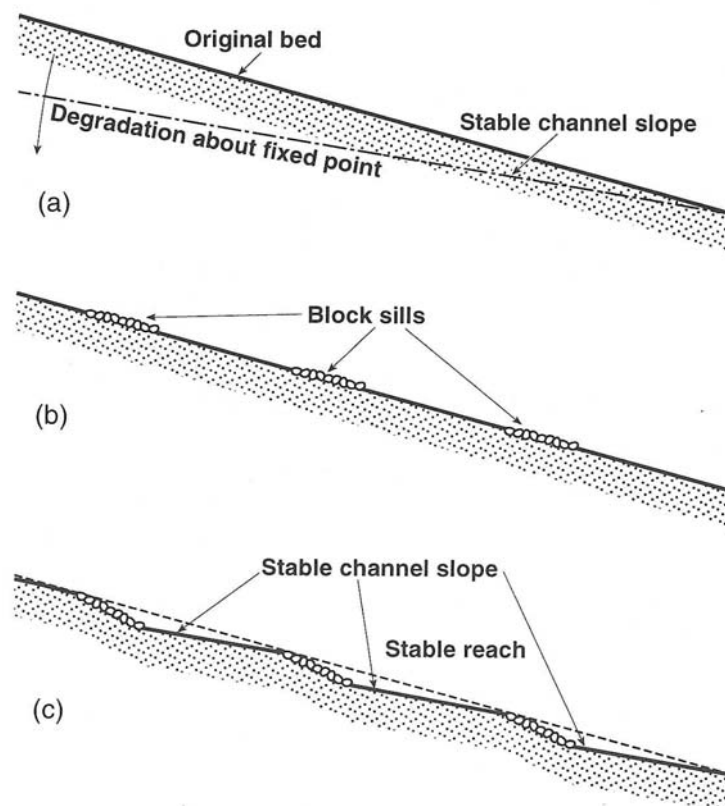


Figure 3.1: Channel stabilization with rock sills (adapted from Whitaker and Jäggi (1986))

Construction of bed sills is sometimes accomplished by placing the rock along the stream bed to act as a hard point to resist the erosive forces within the degradational zone. In other situations, a trench may be excavated across the stream bed and then filled with rock. A critical component in the design of these structures is ensuring that there is a sufficient volume of non-erodible material to resist the general bed degradation, as well as any additional local scour at the structure. This is illustrated in Figure 3.2, which shows a riprap grade control structure designed to resist both the general bed degradation of the approaching knickpoint as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass (layer thickness) to launch into the anticipated scour hole depth.

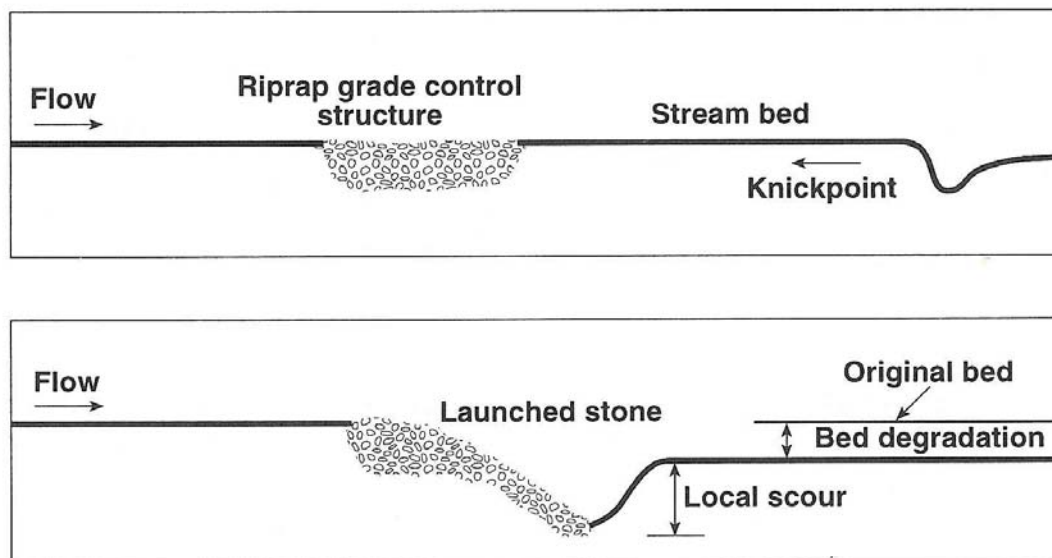


Figure 3.2: The upper drawing is a riprap grade control structure with sufficient launch stone included to satisfy anticipated scour. The lower drawing indicates launching of riprap at grade control structure in response to bed degradation and local scour

A unique type of loose-rock structure is used by Newbury and Gaboury (1993). The structures are placed at 5 to 7 channel width spacing to emulate the spacing of natural riffles. For the Mink Creek example shown in Figure 3.3, the structures were designed to a height of 0.6 m that would impound shallow pools for passage of young walleye fry. No cutoff walls or filters were used in this installation, but the structure was sealed by infilling the front slope with shale gravel scraped from the bed.

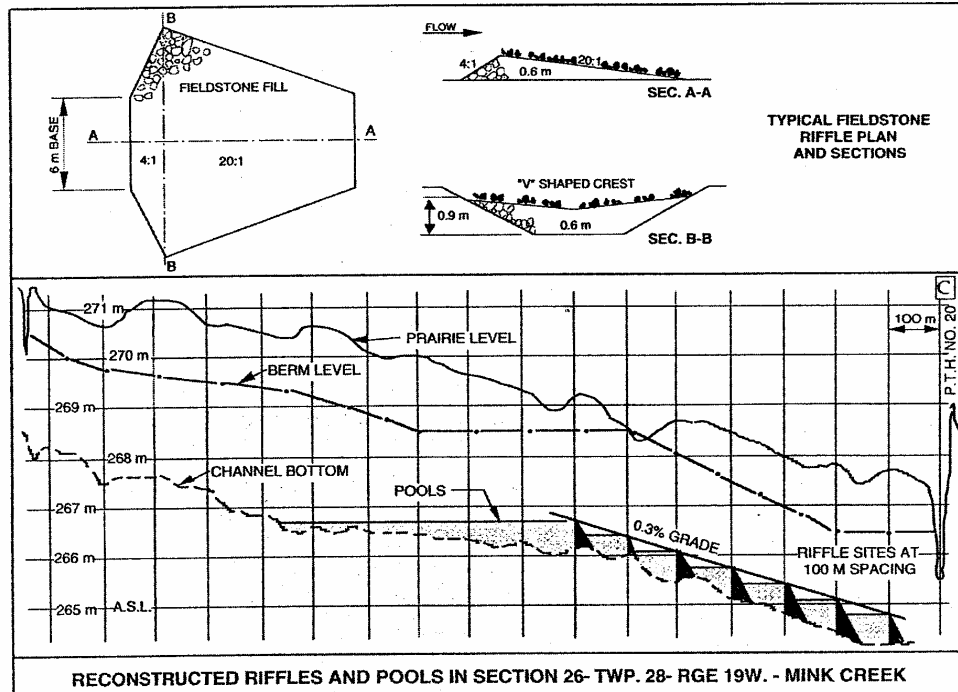


Figure 3.3: Loose-rock structures are show in plan and profile (from Newbury (2002))

Rosgen (2002a) described and provided design specifications for cross-vane, and J-hook vane structures. The purposes of these structures include establishment of grade control and enhancement of fish habitat. Figures 3.4 and 3.5 are from Rosgen (2002a) depicting the construction dimensions of the structures.

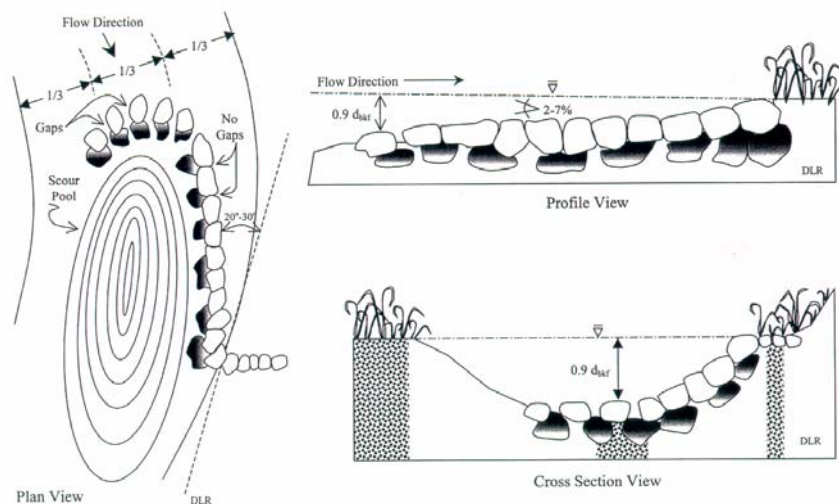


Figure 3.4: Plan, profile, and section view of J-hook structure (Rosgen 2002a)

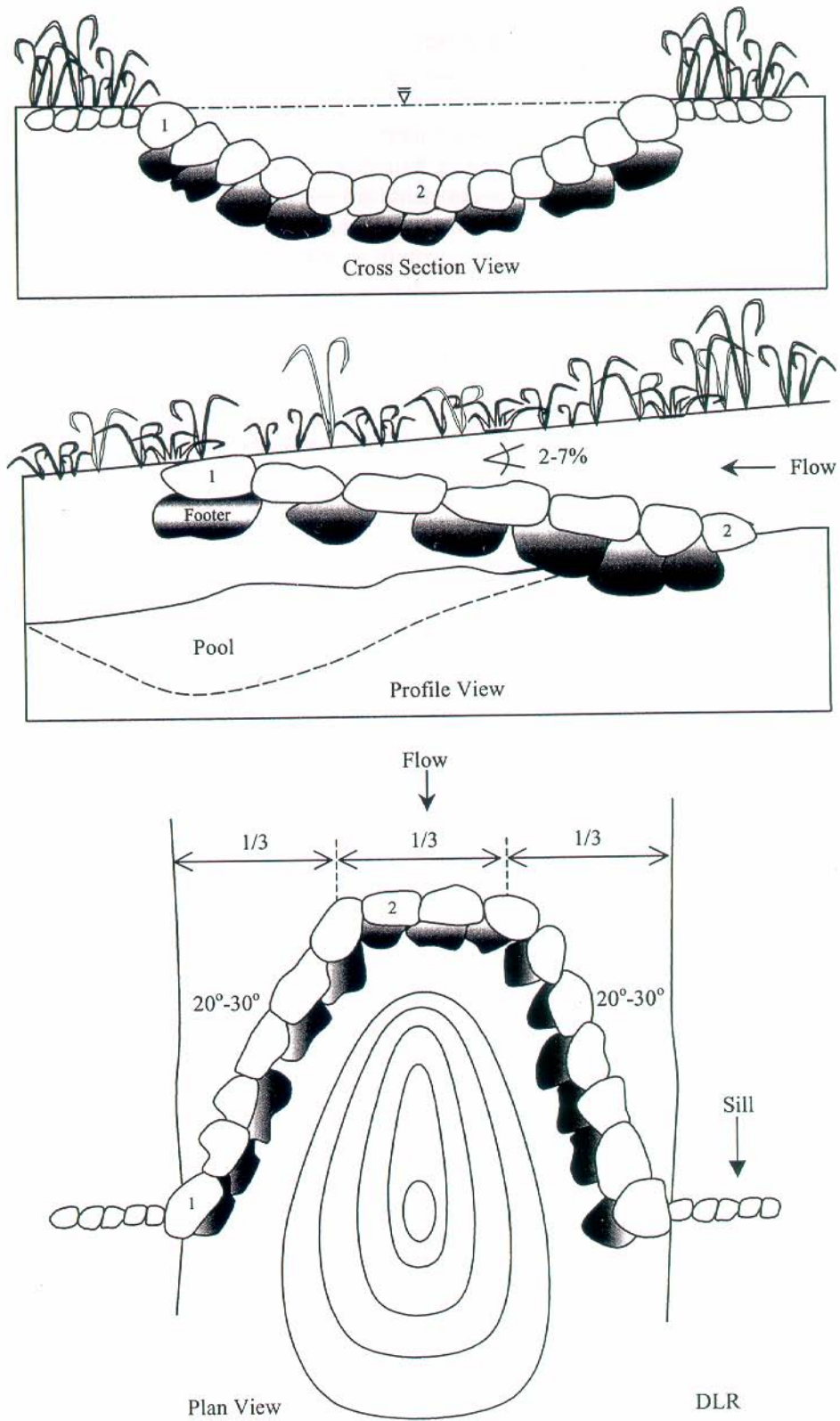


Figure 3.5: Plan, profile, and section view of cross-vane structure (Rosgen 2002a)

3.1.1.1 Rock Sizing for Loose-rock Structures

A common factor in all loose-rock structures is determining the proper size of the stone. Five methods are presented:

- Abt and Johnson (1991)

Abt and Johnson (1991) conducted near-prototype flume studies to determine riprap stability subjected to overtopping flows, for example, in spillway flow or in sloping loose-rock grade control structures. Riprap design criteria for overtopping flows were developed for two conditions: stone movement and riprap layer failure. The criteria were developed as a function of stone shape, median stone size, unit discharge, and embankment slope. Stone movement occurred at approximately 74% of layer failure. It was determined from testing that rounded stone fails at a unit discharge approximately 40% less than angular stone, for the same median size of stone.

The resulting equations for angular riprap developed by Abt and Johnson (1991) are:

$$q_{\text{design}} = \frac{q_f}{0.74} = 1.35q_f \quad (3.1)$$

$$D_{50} = 5.23S^{0.43} q_{\text{design}}^{0.56} \quad (3.2)$$

where: q_f = stone size at failure (in.);
 q_{design} = design discharge (cfs/ft); and
 S = slope of the riprap layer.

- Whittaker and Jäggi (1986)

$$\frac{q}{\sqrt{gD_{65}^3(s-1)}} \leq \frac{0.257}{J^{7/6}} \quad (3.3)$$

where: q = specific discharge over the ramp ($\text{m}^3/\text{s}\cdot\text{m}$);
 D_{65} = characteristic block diameter of the block mixture (m);
 s = specific gravity of the blocks compared to that of the water (e.g., 2.65);

J = ramp gradient; and
g = acceleration due to gravity (m/s²).

- Newbury and Gaboury (1993)

$$\text{tractive force (kg/m}^2\text{)} = \text{incipient diameter (cm)} \quad (3.4)$$

- Robinson *et al.* (1998)

A two-part prediction equation was developed by Robinson *et al.* (1998) to determine the highest stable discharge as a function of the median rock size and channel slope, created for SI units:

$$q = 9.76 \times 10^{-7} D_{50}^{1.89} S_o^{-1.50} \text{ for } S_o < 0.10 \quad (3.5)$$

$$q = 8.07 \times 10^{-6} D_{50}^{1.89} S_o^{-0.58} \text{ for } 0.10 < S_o < 0.4 \quad (3.6)$$

where: q = unit discharge (m³/s/m);

S_o = slope (m/m); and

D₅₀ = median grain size (mm).

- Rosgen (2002a)

$$\text{minimum rock size (m)} = 0.1724 \text{ LN (bankfull shear stress, kg/m}^2\text{)} + 0.6349 \quad (3.7)$$

The Rosgen (2002a) relationship was developed to determine minimum size of rock for the cross-vane and J-hook structures at bankfull flow conditions, and he limits the application of this relationship to river discharge ranging from 0.56 cumecs to 113.3 cumecs, and bankfull depth from 0.26 m. to 1.5 m. It may be implied from the emphasis on bankfull conditions that the Rosgen structures would not be suitable for incised channels that may contain much greater flow than the bankfull flow described by Leopold *et al.* (1964) as the 1.5-year recurrence interval discharge.

Another important point to consider is that the Rosgen (2002a) relationship determines the minimum size rock to be placed in a flowing river, whereas the remaining relationships predict a riprap size to be used in construction of a sloping loose-rock structure. If the sloping loose-rock structures are to be constructed in a location that will encounter completely submerged conditions, then traditional riprap sizing methods (USACE 1991; FHWA 2001b) should be used to check structure stability.

Figure 3.6 compares the five different procedures using a 5% sloping loose-rock structure, at a unit discharge varying from 1 to 10 cubic meters per meter of width. Figure 3.7 portrays similar information, indicating rock size (mm) as a function of tractive force. In addition, an estimate of rounded-stone size (Abt and Johnson 1991) and the Rosgen (2002a) estimate for his structures are shown.

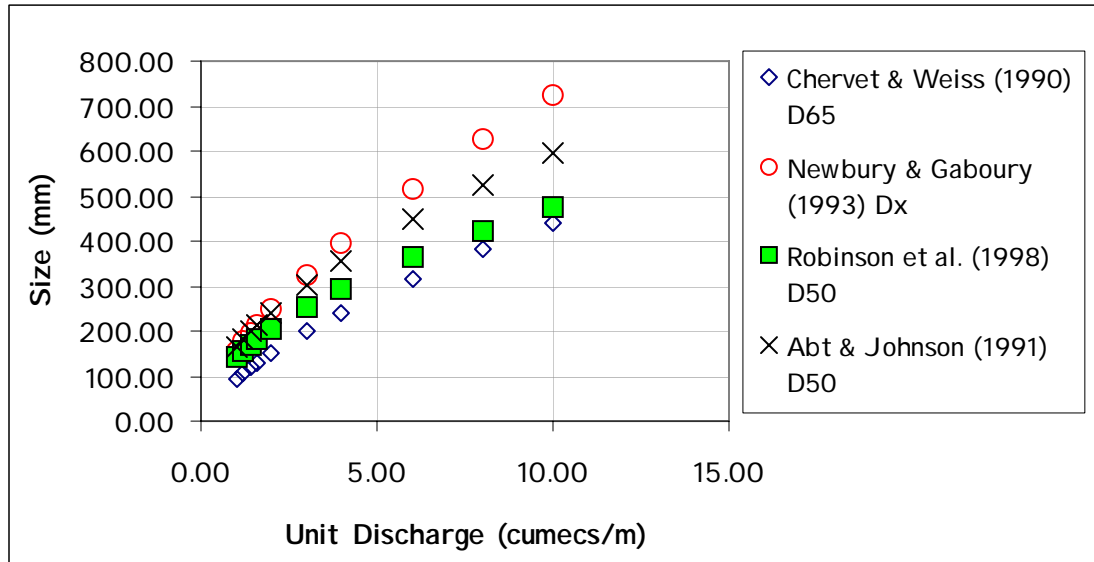


Figure 3.6: Comparison of rock sizing methods for a 1 to 20 sloping face structure

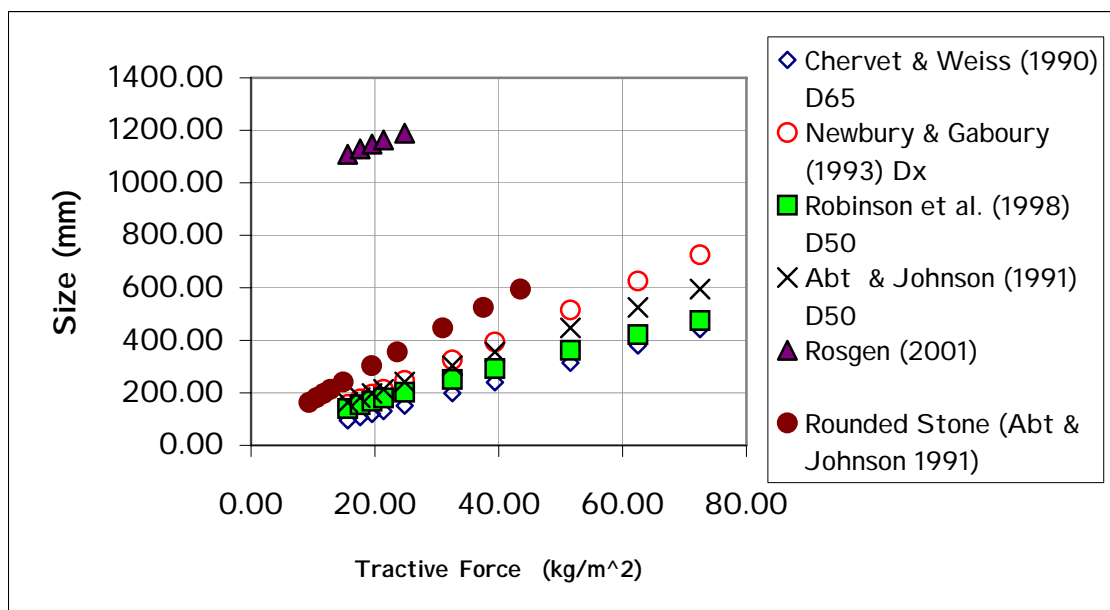


Figure 3.7: Comparison of four rock size selection criteria based on computed shear stress

Newbury and Gaboury (1993) did not specify a rock size within the gradation; for example, Chervet and Weiss (1990) specified D_{65} . As shown by the comparison graph, three of the four methods are in approximate agreement. The Abt and Johnson (1991) and Newbury and Gaboury (1993) methods are the more conservative. The Newbury and Gaboury (1993) method is well within a reasonable gradation range with other methods, for example, the D_{84} size for the Abt and Johnson (1991) method D_{50} , even for a relatively tight gradation coefficient, $G = 2$ (Simons and Sentürk 1992).

3.1.1.2 Local Scour Prediction for Loose-rock Structures

Chervet and Weiss (1990) reviewed work by Whittaker and Jäggi (1986) and developed a relationship for predicting local scour at the downstream extent of a loose-rock structure, referred to by the authors as a block ramp.

The maximum scour depth (t) can be estimated using the following approach (Tschopp-Bisaz, modified in accordance with Whittaker and Jäggi (1986)):

$$t + h_U \cong 0.85q^{0.5} \left(\frac{q}{h_N} \right)^{0.5} - 7.125d_{90} \quad (3.8)$$

where: h_U = tailwater depth (m); and

h_N = normal supercritical discharge depth over the ramp (m), e.g., calculated according to Strickler's formula using a coefficient of friction of $k = 21/D_{65}^{1/6}$ ($m^{1/3}/s$); and

t = predicted scour depth (m).

Local scour depth is directly related to unit discharge, and an inverse relationship is shown for tailwater depth, the D_{90} of the bed material. They recommend that the downstream extent of the structure should extend below an anticipated local scour depth.

Bitner (2003) reviewed local scour depth, reporting that Castro (1999) defined bed key depth as the local scour depth to which the rock structure should be excavated to prevent undermining. Castro (1999) recommended that the scour depth may approach 2.5, the drop height for gravel or cobble beds, and 3.5 times the drop height for sand beds.

3.1.1.3 Channel Linings

Grade control can also be accomplished by lining the channel bed with a non-erodible material. These structures are designed to ensure that the drop is accomplished over a specified reach of the channel, which has been lined with riprap or some other non-erodible material. Rock riprap gradient control structures have been used by the Soil Conservation Service (SCS) for several years (SCS 1976). These structures are designed to flow in the subcritical regime with a constant specific energy at the design discharge, which is equal to the specific energy of flow immediately upstream of the structure (Myers 1982). Although these structures have generally been successful, there have been some associated local scour problems. This precipitated a series of model studies to correct these problems and to develop a design methodology for these structures (Tate 1988, 1991). A plan and profile drawing of the improved structure is shown in Figure 3.8.

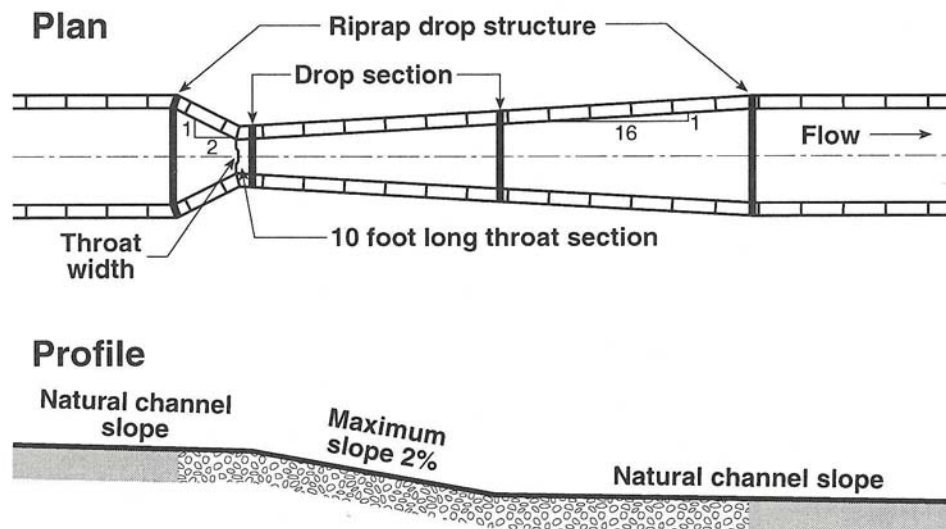


Figure 3.8: Riprap-lined drop structures (adapted from Tate (1991))

3.1.2 Loose-rock Structures with Water Cutoff

One problem often encountered with the above structures is the displacement of rock (or rubble, etc.) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the channel is composed primarily of pervious material. This problem

can be eliminated by constructing a water barrier at the structure. One type of water barrier consists of simply placing a trench of impervious clay fill upstream of the weir crest. This type of water barrier is illustrated in Figure 3.9. In general, this type of barrier has limited longevity due to susceptibility to erosion, which can be avoided by using concrete or sheet piling for the cutoff wall. The conceptual design of a riprap grade control structure with a sheet pile cutoff wall is shown in Figure 3.10.

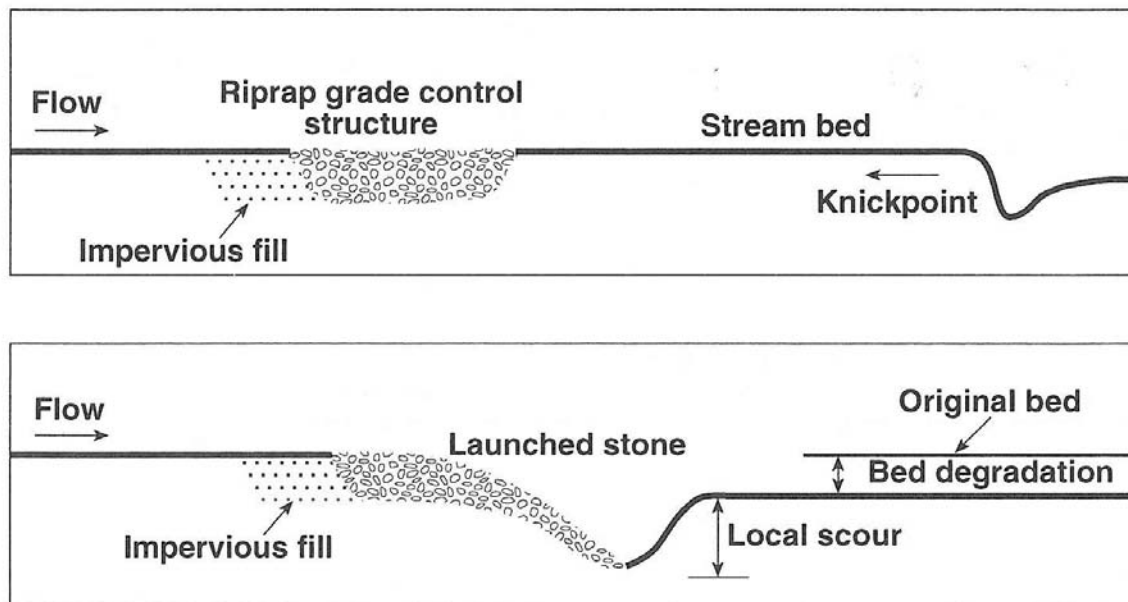


Figure 3.9: The upper drawing is a built riprap grade control structure with impervious fill cutoff wall, and the lower drawing indicates launching of riprap at grade control structure in response to bed degradation and local scour

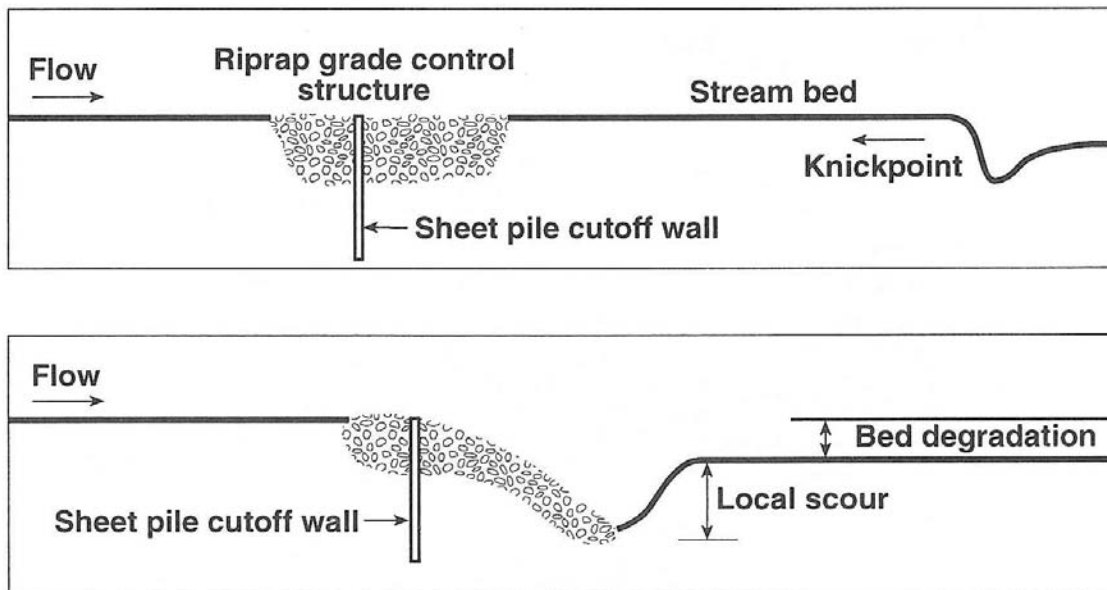


Figure 3.10: The upper drawing is a built riprap grade control structure with sheet pile cutoff wall, and the lower drawing indicates launching of riprap at grade control structure in response to bed degradation and local scour

3.1.3 Structures with Pre-formed Scour Holes and Water Cutoff

A scour hole is a natural occurrence downstream of any overfall. Sizing of the scour hole is a critical element in the design process, which is usually based on model studies or on experience with similar structures in the area.

The stability of rock structures is often jeopardized at low tailwater conditions. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. Linder (1963) developed a structure that is designed to operate at submerged conditions where the tailwater elevation (T) does not fall below 0.8 of the critical depth (D_c) at the crest section (Linder 1963). Subsequent monitoring of the in-place structures confirmed the successful performance in the field (USACE 1981).

Little and Murphey (1982) developed a loose-rock structure, incorporating a sheet pile cutoff and weir, a preformed scour basin lined with riprap that acts as an energy dissipation basin. They observed that an undular hydraulic jump occurs when the incoming Froude number is less than 1.7. Consequently, Little and Murphey developed a grade control design that included an energy dissipating baffle to breakup these undular waves (Figure 3.11). This

structure, referred to as the Agricultural Research Services (ARS) type low-drop structure, has been used successfully in north Mississippi for drop heights up to about 2 m by both the USACE and the U.S. Department of Agriculture (USDA) SCS (USACE 1981). A recent modification to the ARS structure was developed following model studies at Colorado State University (CSU) (Johns *et al.* 1993; Abt *et al.* 1994). The modified ARS structure, presented in Figure 3.12, retains the baffle plate but adopts a vertical drop at the sheet pile rather than a sloping rock-fill section.

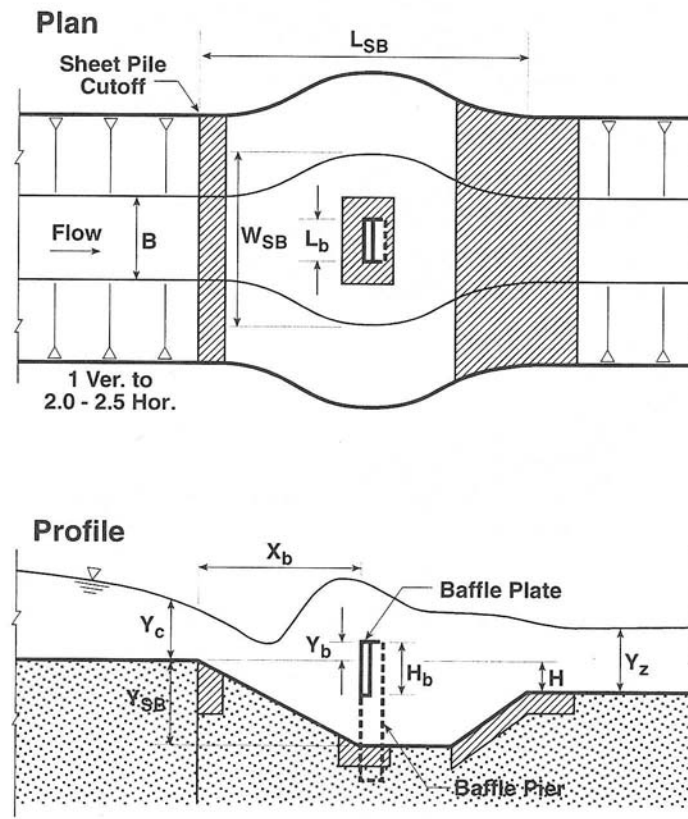


Figure 3.11: ARS-type grade control structure with pre-formed riprap lined stilling basin and baffle plate (adapted from Little and Murphey (1982))

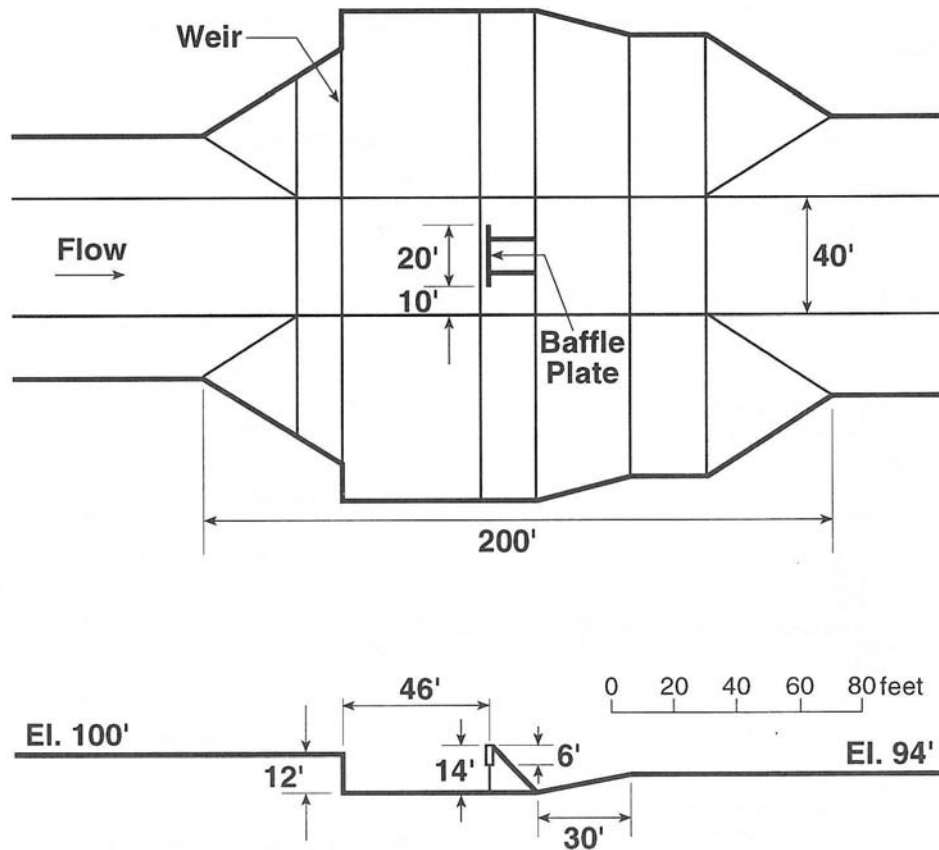


Figure 3.12: Schematic of modified ARS-type grade control structure (Abt *et al.* 1994)

Smith and Wilson (1992) provide guidance for design and construction of the ARS-type grade control structure. The guidance is replete with information; several specific points follow:

- For selection of the final structure site, the channel should be straight for a distance of 10 channel widths upstream and for a minimum of 200 ft downstream.
- No gullies or lateral drains should occur in the site.
- The base width of the weir should be constricted to ensure that the water surface elevation of the 2-year discharge moves from critical depth near the weir crest to normal depth of flow in a short distance, for example, a few channel widths.
- The resulting flood-control impacts should not violate flood-control requirements.
- The stilling basin dimensions should be based on the smaller of the bankfull discharge or the 100-year discharge.

- Downstream tailwater conditions should be based on normal depth calculations of an estimated future, degraded condition.
- Stilling basin riprap size is based on physical model studies referenced in the guidance. Approach channel protection and exit channel protection are specified.

3.1.4 Rigid-drop Structures

In many situations where the discharge and/or drop heights are large, in excess of 2 m, grade control structures are frequently constructed of concrete, or a combination of sheet pile and concrete. There are many different designs for concrete grade control structures. The two discussed herein are the California Institute of Technology (CIT) and the St. Anthony Falls (SAF) structures. Both of these structures were utilized on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and channel conditions (Stufft 1965). Where the discharges were large and the channel depth was relatively shallow, the CIT type of drop structure was utilized. The CIT-type structure is generally applicable to low-drop situations where the ratio of the drop height to critical depth is less than 1; however, for the Gering Drain project this ratio was extended up to 1.2. The original design of this structure was based on criteria developed by Vanoni and Pollack (1959). The structure was then modified by model studies at the Waterways Experiment Station (WES) in Vicksburg, Mississippi, and is shown in Figure 3.13 (Murphy 1967). Where the channel was relatively deep and the discharges smaller, the SAF-type drop structure was used. This design was developed from model studies at the SAF Hydraulic Laboratory for the SCS (Blaisdell 1948). This structure is shown in Figure 3.14. The SAF-type structure is capable of functioning in flow situations where the drop height to critical depth ratio is greater than 1 and can provide effective energy dissipation within a Froude number range of 1.7 to 17. Both the CIT and the SAF drop structures have performed satisfactorily on the Gering Drain for over 25 years.

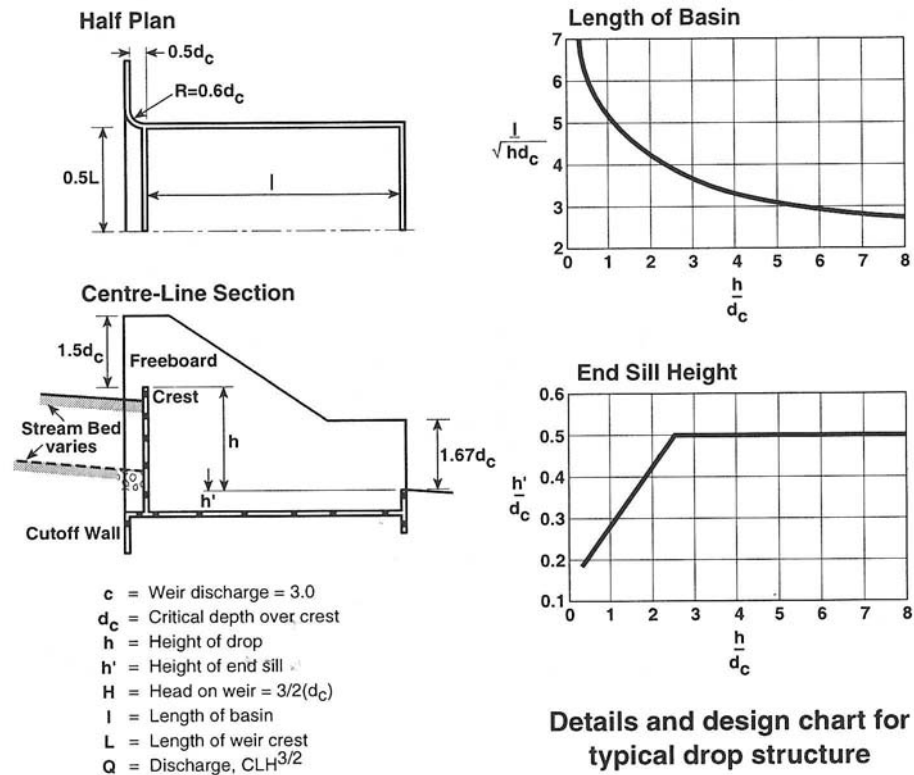


Figure 3.13: CIT-type drop structure (Murphy 1967)

The design for a large rigid structure should include consideration of slope stability, including sudden drawdown, for the site and for approach and downstream channels. Stability analyses should include sliding stability of the structure, underseepage, and allowance for bearing capacity and settlement. As the hydraulic capacity and drop height of the structure increases, the complexity of design and construction increases.

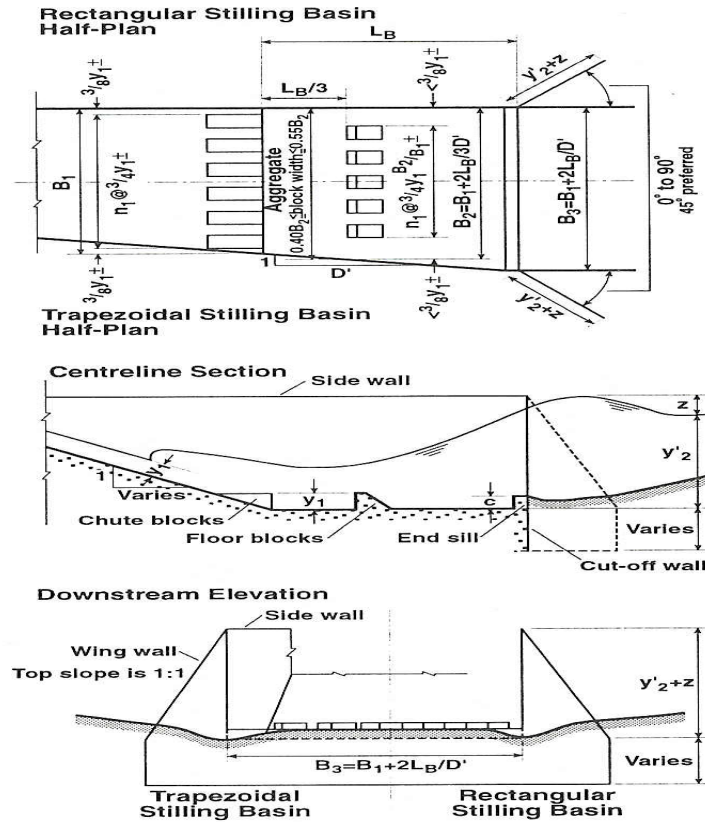


Figure 3.14: SAF type drop structure (Blaisdell 1948)

3.2 ALTERNATIVE CONSTRUCTION MATERIALS

While riprap, sheet pile, and concrete may be the most commonly used construction materials for grade control structures, there are many situations where cost or availability of materials may prompt the engineer to consider other alternatives. Gabion grade control structures are often an effective alternative to standard riprap or concrete structures (Hanson *et al.* 1986). Guidance for the construction of gabion weirs is also provided by the USACE (1974). Gabion mattresses consist of rectangular-shaped wire mesh baskets filled with rock (FHWA 1989). Current applications of gabion mattresses include streambed and bank stability. Table 3.1 presents the advantages and disadvantages of gabion mattresses when utilized in an erosion control application. Other, more detailed design guidelines for rock gabions can be found in FHWA (1989), USACE (1991), and Maynard (1995).

Table 3.1: Advantages and disadvantages of rock-filled gabion mattresses (adapted from FHWA (1989))

Advantages	Disadvantages
Ability to span minor pockets of subsidence without failure	Susceptibility of the wire baskets to corrosion and abrasion damage
Interlock to allow use of smaller, lower quality rock in the baskets	High labor cost associated with fabrication and filling the baskets
Economically feasible where riprap sized rock is not readily available	More difficult and expensive repair than standard rock protection

Bitner (2003) pointed out that an alternative to the conventional riprap or concrete structure that has gained popularity in the southwestern United States is the use of soil cement grade control structures. These structures are constructed of on-site soil-sand in a mix with Portland Cement to form a high quality, erosion-resistant mixture. Soil cement grade control structures are most applicable when used as a series of small drops in lieu of a single large-drop structure. Experience has indicated that a limiting drop height for these structures is on the order of 1 m. Design criteria for these structures are presented by Simons and Li (1982).

Thornton *et al.* (1999) has developed shear resistance criteria for A-Jacks, an interlocking concrete armor unit manufactured by Armortec Erosion Control Solutions, Inc. (Figure 3.15). Current applications of A-Jacks include coastal shoreline protection, stream bed and bank protection, and pier scour mitigation. Depending on their intended application, A-Jacks vary between 2 to 8 ft in size as summarized in Table 3.2.

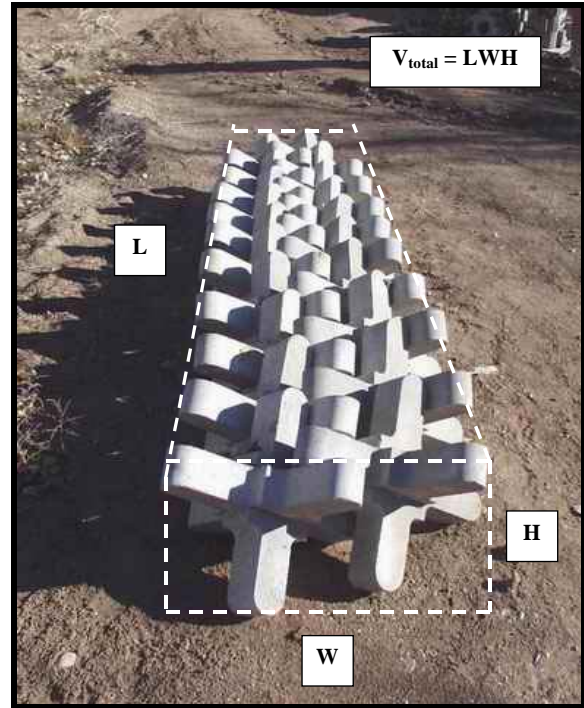
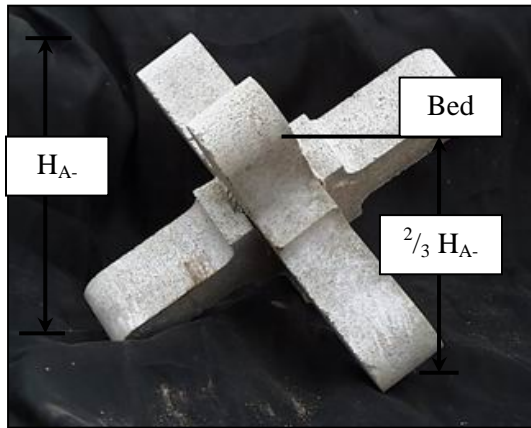
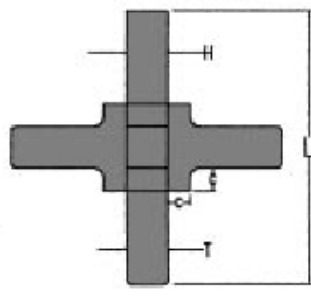


Figure 3.15: A-Jacks are frequently used in an interlocking matrix to maximize shear resistance

Table 3.2: A-Jacks dimensions from the Armortec Erosion Control Solutions, Inc. website (2002)

A-Jacks	L(in)	T(in)	H(in)	C(in)	Vol(ft ³)	Wt(lbs)
AJ-24	24	3.68	3.68	1.84	0.56	78
AJ-36	36	5.52	5.52	2.76	1.89	265
AJ-48	48	7.36	7.36	3.68	4.49	629
AJ-72	72	11.04	11.04	5.52	15.14	2,120
AJ-96	96	14.72	14.72	7.36	35.87	5,022

AJ-24 and AJ-36 are best used for streambank restoration. AJ-48, AJ-72 and AJ-96 are intended for energy dissipation and ocean applications.



*Actual dimensions may vary

Bitner (2003) reported that due to the porous nature of an A-Jacks matrix, a filter design should be incorporated in most instances. FHWA (2001b) presents methods of incorporating rock filter design for use with A-Jacks on bridge pier scour applications. Bitner (2003) conducted extensive physical model testing of A-Jacks grade control structures.

Stone riprap can be bound with cement grout, forming grouted riprap. The apparent advantage in grouted riprap is to increase the shear resistance of individual stone particles. In their review of grouted riprap, Przedwojski *et al.* (1995) cited three basic methods of grouting (Rijkswaterstaat 1985):

- 1) Surface grouting fills approximately 30% of the surface voids, with mortar penetrating the surface layer without completely sealing the construction.
- 2) Pattern grouting fills 50% to 80% of cover-layer voids, and penetrates the full thickness of the riprap. Eventually, a mesh of stone-cement aggregates is formed.
- 3) Full grouting fills 100% of the cover-layer voids, resulting in an impermeable layer.

They caution that as voids are filled with grout and permeability diminishes, the stability of the layer is adversely affected by excess pore pressures occurring during high discharges or from groundwater. Weepholes or other positive drainage should be provided to avoid massive failure.

McLaughlin Water Engineers (1988) report that grout has been successfully used to stabilize loose riprap, many failures have been reported that were associated with seepage and uplift. They recommend that seepage be controlled by constructing a vertical cutoff immediately upstream of the crest, constructing the cutoff by excavating a trench below the riprap subgrade and placing steel and concrete to form the cutoff wall. Their view of grouted riprap is different from Przedwojski *et al.* (1995). McLaughlin Water Engineers (1986) recommends that regular riprap should not be used with grout, and that rock with all dimensions greater than the grout thickness be required and placed on a firm subgrade. Grout is then pumped into the voids and vibrated filling the voids between rocks. The method results in the appearance of a concrete slab with large stones spaced evenly, protruding through the slab. Toe and lateral drains are included for drainage of the grouted area.

3.3 DOWNSTREAM CHANNEL RESPONSE

Since grade control structures affect the sediment delivery to downstream reaches, it is necessary to consider the potential impacts to the downstream channel when grade control structures are planned. Bed control structures reduce the downstream sediment loading by preventing the erosion of the bed and banks, while hydraulic control structures have the added effect of trapping sediments. The ultimate response of the channel to the reduction in sediment

supply will vary from site to site. In some instances, the effects of grade control structures on sediment loading may be so small that downstream degradational problems may not be encountered. However, in some situations such as when a series of hydraulic control structures is planned, the cumulative effects of sediment trapping may become significant. In these instances, it may be necessary to modify the plan to reduce the amount of sediment being trapped or to consider placing additional grade control structures in the downstream reach to protect against the induced degradation. Therefore, following the hydraulic spacing of a series of grade control structures using a thorough investigation of providing a balance between supply and transport of water and sediment, the designer must utilize a long-term sediment routing model, to investigate downstream channel response.

3.4 GEOTECHNICAL CONSIDERATIONS

The above discussions focused only on the hydraulic aspects of design and siting of grade control structures. However, in some cases, the geotechnical stability of the reach may be an important or even the primary factor to consider when siting grade control structures. This is often the case where channel degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank height (Thorne and Osman 1988). When this occurs, bank instability may be widespread throughout the system rather than restricted to the concave banks in bendways. Traditional bank-stabilization measures may not be feasible in situations where system-wide bank instabilities exist. In these instances, grade control, aimed at preventing the onset of incision-triggered mass-wasting, may be the more appropriate solution.

Grade control structures can enhance the bank stability of a channel in several ways. Bed control structures indirectly affect the bank stability by stabilizing the bed, thereby reducing the length of bankline that achieves an unstable height. Two advantages of grade control structures with respect to bank stability are: 1) bank heights can be reduced due to sediment deposition upstream of the structure increasing bank stability; and 2) scouring potential can reduce or eliminate the severity and extent of basal cleanout of the failed bank material, thereby promoting self healing of the banks (Thorne 1990). Additional references pertaining to stream-bank stability include: American Society of Civil Engineers (ASCE 1998); Bishop (1955); Coppin and Richards (1990); Gray and Leiser (1982); Hagerty (1990); Huang (1983); Kouwen *et al.* (1969);

López and Garcia (1997); Morgenstern and Price (1965); Osman and Thorne (1988); Sands and Kapitzke (1998); Simon *et al.* (1991, 1999); Terzaghi (1943); and Terzaghi and Peck (1967).

The flow of water through a pervious foundation can be a serious problem for a grade control structure. As the drop height of the structure increases, the driving force for subsurface flow and possible erosion beneath the structure increases. Very silty and sandy soils are the least resistant to seepage or piping failures (McLaughlin Water Engineers 1986). Seepage pressures and velocities must be controlled to prevent internal erosion and particle migration. In extreme cases, seepage may cause failure of the structure foundation and sloughing of the stream bank downstream of the crest of the structure. General discussion of seepage theory and analysis is provided in Cedergren (1977), and embankment flownets are discussed in depth by Sherard *et al.* (1963) and Vlope and Kelly (1985), as referenced in Novak *et al.* (1997).

Common methods of seepage control include cutoff trenches filled with an impervious material, sheet pile curtains, upstream impervious blankets, and downstream filter blankets. The USBR (1987) provides an intensive discussion of these methods.

The rational approach to design of filters is generally credited to Terzaghi and Peck (1967), and additional experimentation has been performed by the USACE (1941) and USBR (1955). The purpose of the graded filter is to provide a permeable layer to permit seepage flow that relieves seepage uplift forces without allowing soil particles from being washed from the foundation and washed into the filter and clogging the filter (USBR 1987). Filter specifications for seepage control or for underlying riprap (Simons and Sentürk 1992; USACE 1991) are essentially the same.

$$\frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5 < \frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40 \quad (3.9)$$

and

$$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40 \quad (3.10)$$

where: D_x = particle size for which x percent by weight are finer (ft);
filter = overlying material layer; and
base = underlying material layer.

In order to obtain the above criteria, some designs may require multiple filter layers increasing in gradation from bottom to top. Filter layer thickness varies from 6 to 15 inches for single layers, and 4 to 8 inches on multiple layers (Chang 1998).

3.5 FLOOD CONTROL IMPACTS

Channel improvements for flood control and channel stability often appear to be mutually exclusive objectives. For this reason, it is important to ensure that any increased post-project flood potential is identified. This is particularly important when hydraulic control structures are considered. In these instances, the potential for causing overbank flooding may be the limiting factor with respect to the height and amount of constriction at the structure. Grade control structures are often designed to be hydraulically submerged at flows less than bankfull so that the frequency of overbank flooding is not affected. However, if the structure exerts control through a wider range of flows including overbank, then the frequency and duration of overbank flows may be impacted. When this occurs, the impacts must be quantified and appropriate provisions such as acquiring flowage easements or modifying structure plans should be implemented.

Another factor that must be considered when designing grade control structures is the safe return of overbank flows back into the channel. This is particularly a problem when the flows are out of the bank upstream of the structure but still within the bank downstream. The resulting head differential can cause damage to the structure as well as severe erosion of the channel banks depending on where the flow re-enters the channel. Some means of controlling the overbank return flows must be incorporated into the structure design. One method is simply to design the structure to be submerged below the top bank elevation, thereby reducing the potential for a head differential to develop across the structure during overbank flows. If the structures exert hydraulic control throughout a wider range of flows including overbank, then a more direct means of controlling the overbank return flows must be provided. One method is to ensure that all flows pass only through the structure. This may be accomplished by building an earthen dike or berm extending from the structure to the valley walls that prevents any overbank

flows from passing around the structure (Forsythe 1985). Another means of controlling overbank flows is to provide an auxiliary high-flow structure which will pass the overbank flows to a specified downstream location where the flows can re-enter the channel without causing significant damage (Hite and Pickering 1982).

3.6 ENVIRONMENTAL CONSIDERATIONS

Projects must work in harmony with the natural system to meet the needs of the present, without compromising the ability of future generations to meet their needs. Engineers and geomorphologists are responding to this challenge by trying to develop new and innovative methods for incorporating environmental features into channel projects. The final siting of a grade control structure is often modified to minimize adverse environmental impacts to the system.

Grade control structures can provide direct environmental benefits to a stream. Cooper and Knight (1987) conducted a study of fisheries resources below natural scour holes and manmade pools below grade control structures in northern Mississippi. They concluded that although there was greater species diversity in the natural pools, there was increased growth of game fish and a larger percentage of harvestable-size fish in the manmade pools. They also observed that the manmade pools provided greater stability of reproductive habitat. Shields *et al.* (1990) reported that the physical aquatic habitat diversity was higher in stabilized reaches of Twentymile Creek, Mississippi, than in reaches without grade control structures. They attributed the higher diversity values to the scour holes and low-flow channels created by the grade control structures. The use of grade control structures as environmental features is not limited to the low-gradient sand bed streams of the southeastern United States. Jackson (1974) documented the use of gabion grade control structures to stabilize a high-gradient trout stream in New York. She observed that following construction of a series of bed sills, there was a significant increase in the density of trout. The increase in trout density was attributed to the accumulation of gravel between the sills, which improved the spawning habitat for various species of trout.

Perhaps the most serious negative environmental impact of grade control structures is the obstruction to fish passage. In some cases, particularly when drop heights are small, fish are able to migrate upstream past a structure during high flows (Cooper and Knight 1987). However, in

situations where structures are impassable, and where the migration of fish is an important concern, openings, fish ladders, or other passageways must be incorporated into the design of the structure to address fish-movement problems (Nunnally and Shields 1985). The various methods of accomplishing fish movement through structures are not discussed here. Interested readers are referred to Nunnally and Shields (1985), Clay (1961), and Smith (1985) for more detailed discussions.

The environmental aspects of the project must be an integral component of the design process when siting grade control structures. A detailed study of all environmental features in the project area should be conducted early in the design process. This will allow these factors to be incorporated into the initial plan rather than having to make costly and often less environmentally effective last-minute modifications to the final design. Unfortunately, there is very little published guidance concerning the incorporation of environmental features into the design of grade control structures. A source of useful information can be found in the following technical reports published by the Environmental Laboratory of the USACE, WES: Shields and Palermo (1982), Henderson and Shields (1984), and Nunnally and Shields (1985).

The above discussion illustrates that the design of grade control structures is not simply a hydraulic exercise. Rather, there are many other factors that must be included in the design process. For any specific situation, some or all of the factors discussed in this section may be critical elements in the final siting of grade control structures. It is recognized that this does not represent an all-inclusive list since there may be other factors not discussed here that may be locally important. For example, in some cases, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive cookbook procedure for designing grade control structures that can be applied universally. However, consideration of each factor in an analytic and balanced fashion, and avoiding reliance on empirical procedures, can lead to effective and intelligent use of grade control structures.

3.7 INVESTIGATION OF MODELS FOR SITING AND DESIGN OF GRADE CONTROL MODELS

Design considerations for improving the effectiveness of grade control structures include determination of the type, location, and spacing of structures along the stream, along with the

elevation and dimensions of structures. Siting grade control structures is often considered a simple optimization of hydraulics and economics. However, hydraulics and economics alone are usually not sufficient to define the optimum spacing for grade control structures. In practice, the hydraulic considerations must be integrated with a host of other factors, which vary from site to site, to determine the final structure plan. Each of these factors should be considered in determining the effectiveness of the structures.

One of the most important steps in the siting of a grade control structure or a series of structures is the determination of the anticipated drop at the structure. This requires some knowledge of the ultimate channel morphology, both upstream and downstream of the structure, which involves assessment of sediment transport and channel morphologic processes.

The hydraulic spacing of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation which, in turn, is a function of the next structure downstream. Heede and Mulich (1973) suggested that the optimum spacing of structures is such that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (1982) showed that the optimum spacing should be the length of the deposition above the structure that is a function of the deposition slope (Figure 3.16). Figure 3.16 also illustrates the recommendations of Johnson and Minaker (1944), that the most desirable spacing can be determined by extending a line from the top of the first structure at a slope equal to the maximum equilibrium slope of sediment upstream until it intersects the original stream bed. However, each of the above references implicitly includes a specific sediment supply concentration, and that concentration is necessary for rational designs.

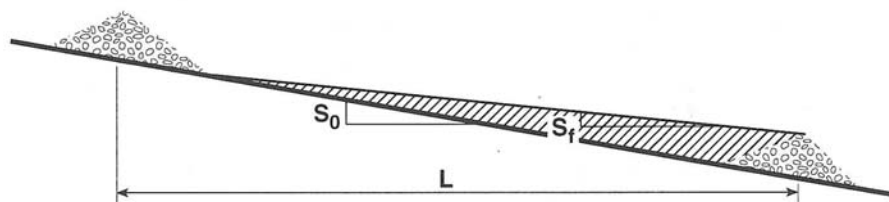


Figure 3.16: Spacing of grade control structure (adapted from Mussetter (1982))

If flow is at normal depth with the bed slope and energy slope equal, as in uniform flow, the hydraulic spacing of grade control structures is straightforward, as shown by the following equation (Goitom and Zeller 1989):

$$H = (S_o - S_f)x \quad (3.11)$$

where: H = amount of drop to be removed from the reach;

S_o = original bed slope and energy;

S_f = final, or equilibrium bed and energy slope; and

x = length of the reach.

The number of structures (N) required for a given reach can then be determined by:

$$N = H/h \quad (3.12)$$

where h is the selected drop height of the structure.

It follows from Equation (3.11), that one of the most important factors when siting grade control structures is the determination of the equilibrium slope (S_f). Failure to properly define the equilibrium slope can lead to costly, overly conservative designs, or inadequate design resulting in continued maintenance problems and possible complete failure of the structures. Clearly, equilibrium slope (S_f) is a function of the sediment supply and is the slope required to transport the sediment supplied. The relationship between the sediment supply and the transport capacity upstream of the structure is strongly affected by the hydraulic characteristics of the structure control section, the extent of initial backwater, and the duration of flows that are controlled.

In reality, the uniform flow assumption would rarely occur in an unstable, degrading channel at the time of grade control structure placement, and prudent design would gradually vary flow assumptions as presented in Section 2.1.

Relationships among parameters of basin geometry, flood discharge, slope, and sedimentary characteristics can be used to develop equilibrium slope design relationships. Hack (1957) proposed the following relationship relating the channel slope (S in ft/mi) to drainage area (A in sq mi) and to the bed material size for which 50% are finer (d_{50} in mm) for streams in Virginia and Maryland:

$$S = a \left(\frac{d_{50}}{A} \right)^b \quad (3.13)$$

In his original paper, Hack (1957) utilized map-measured values of slope, not field-measured data, and found that $b = 0.6$. Schröder (1991) used local slope data from Hack (1957) and reported that $a = 0.0076$ and $b = 0.4$ for units of mm and sq km, respectively. Converting Schröder's analysis to the units originally used by Hack (1957), which are mm and sq mi, the value of $a = 0.00517$ and $b = 0.4$, respectively. In addition to the Virginia and Maryland data used by Hack, Schröder added data from Germany, which when converted to original units of sq mi and mm, yield $a = 0.00449$ and $b = 0.4$, respectively.

Schumm *et al.* (1984) utilized the Channel Evolution Model (CEM) to determine equilibrium reaches (Types IV and V), and Biedenharn (USACE 1990) plotted the slope (S) as a function of drainage area (A in sq mi) to develop Figure 3.17.

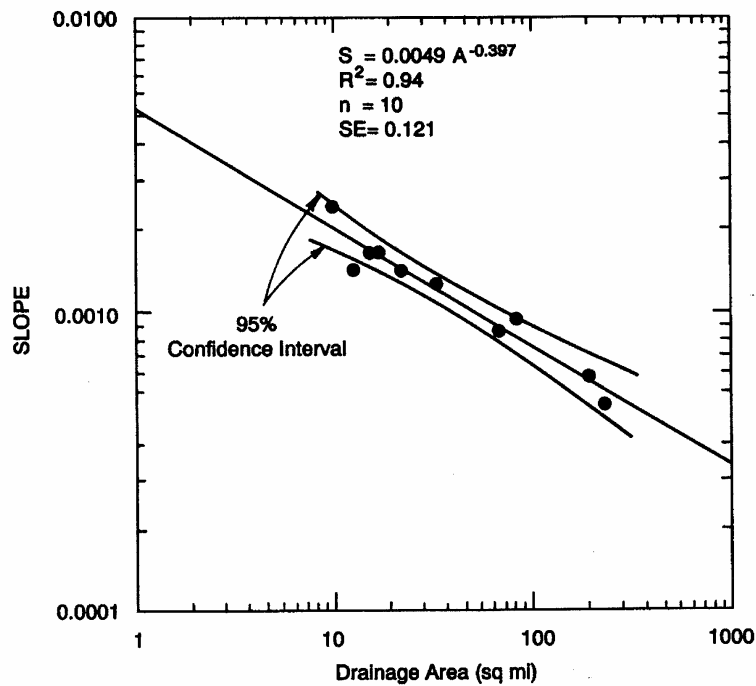
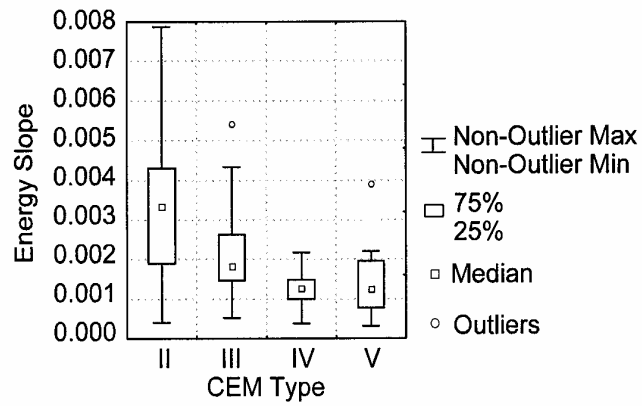


Figure 3.17: Slope-drainage area relationship (from USACE (1990))

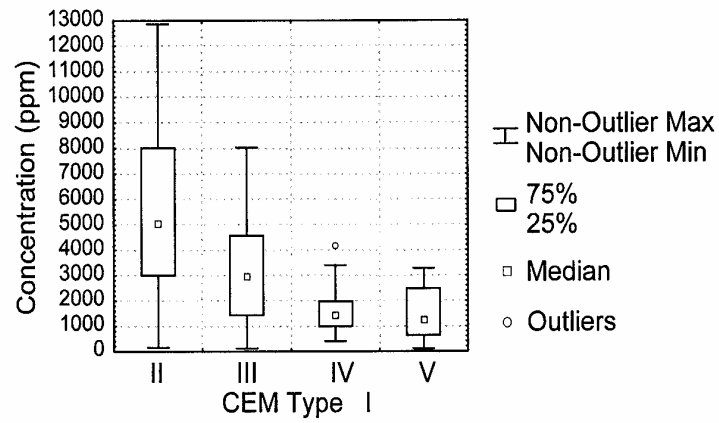
As shown in Figure 3.17, the magnitude of the values of a and b are similar to those produced by the work of Schröder (1991) and Hack (1957). For the figure, d_{50} is assumed to be constant at 0.3 mm.

Another empirical analysis of the Demonstration Erosion Control (DEC) data allows the designer to develop a level of comfort with the data, and perhaps guide the use of more complex models. The sediment transport capacity of each of the 26 Yazoo Basin study reaches was computed by Watson *et al.* (1996). These data indicate that sediment transport capacity decreases as the channel evolves to stability.

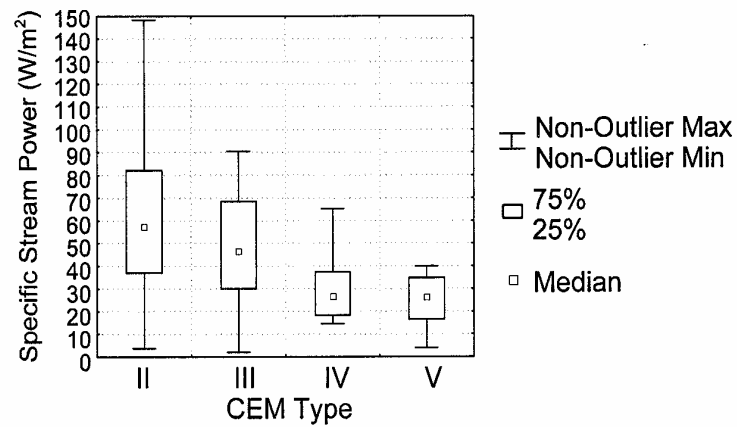
Data on hydraulic characteristics and CEM type from all streams were used to compare trends in slope, sediment transport capacity (shown as concentration), and specific stream power across evolutionary stages for a set of 26 stream reaches. Drainage area for the basins contributing to the 26 stream reaches ranged from 1.3 to 120 sq mi, with a median value of 10.2 sq mi. For these drainages, the 2-year recurrence interval discharge ranged from 6,200 to 269 cubic feet per second (cfs), with a median discharge of 1,369 cfs. The assumption of uniform flow is made for this investigation, allowing the comparison of bed slopes and computed energy slopes. Box and whisker plots are shown in Figure 3.18 for: a) the energy slope, b) the concentration, and c) the specific stream power for all reaches at the 2-year recurrence interval discharge. The concentration was computed in parts per million (ppm) using the Brownlie (1981) sediment transport relationship for the 2-year recurrence interval discharge. The specific stream power was computed as the product of the specific weight of water, the 2-year recurrence interval discharge, and the energy slope, divided by the channel width. Units of specific discharge are in Watts per square meter ($14.56 \text{ W/m}^2 = 1 \text{ ft-lb/sec/ft}^2$). As shown in Figure 3.18, each parameter (slope, concentration, specific stream power) decreases in value from the Type II to the Type V reaches. Box and whisker graphs indicate the median value, a box containing all the data between the 25th and 75th percentiles, and whiskers to the maximum and minimum non-outlier values. Outliers were defined as values that are outside the 25th and 75th percentile range by more than 1.5 times the difference between the 25th and 75th percentiles. This analysis represents the largest combined data set of CEM type and hydraulics analyzed to date. Although there is substantial variation in hydraulic attributes within each evolutionary stage, the expected trends are clearly present. Specific stream power appears to be an excellent predictor of channel stability, with most streams attaining relative stability at specific stream power less than 30 W/m^2 (Figure 3.18c).



(a) Energy Slope



(b) Concentration



(c) Specific Stream Power

Figure 3.18: Box and whisker plots for all DEC reaches

The SAM procedure provides for computation of width, depth, and energy slope; whereas the slope-area method only provides slope. The SAM procedure is flexible, allowing a range of effective discharges and sediment concentrations to be utilized. An important feature of the SAM procedure is that minimum slope for a selected design concentration is computed, which means that degradation between structures and final slope following complete stream and watershed restoration can be predicted (Thomas *et al.* 1994).

Gessler (1993) developed a sensitivity analysis of factors affecting the SAM stable channel computation. For a constant discharge and sediment concentration, he found that, in order of decreasing importance, the d_{50} , the bank angle, and Manning's roughness coefficient (n) are the most important factors in determining stable channel dimensions.

For preliminary design and layout of features in a watershed, an alternative approach based on the SAM model that accounts for additional parameters is suggested. The following relationships are a multi-linear power function of the stable values of slope based on Copeland's (Copeland and Hall 1998) method in SAM. For channels in lower regime with sediment concentrations of 1,000 ppm or less ($R^2 = 0.999$):

$$S = 0.000112 Q^{-0.261} * d_{50}^{0.503} * B^{0.203} * Q_s^{0.631} \quad (3.14)$$

where: S = energy slope (ft/ft);
 Q = discharge (cfs);
 d_{50} = sand diameter (mm);
 B = bank angle expressed in horizontal component of the ratio, i.e., 1 on 3;
 Q_s = sediment transport capacity (ppm); and
Manning's n = 0.035.

For channels in the upper regime with sediment concentrations greater than or equal to 1,500 ppm ($R^2 = 0.916$):

$$S = 0.00241 Q^{-0.301} * d_{50}^{0.711} * B^{0.206} * Q_s^{0.249} \quad (3.15)$$

Caution should be used in applying Equation (3.15) since an erroneous assumption of upper regime flow could result in underestimates of channel roughness and stable channel slope.

Although the equations presented above are the result of over 500 SAM stable channel designs, caution must be used in application to avoid unwarranted extrapolation and erroneous

estimates of channel roughness based on Brownlie's transitional and upper regime thresholds. The range of data used in development of Equations (3.14) and (3.15) is:

Q	=	100 to 10,000 cfs;
d ₅₀	=	0.2 to 0.5 mm uniform sand;
B	=	1:1, 2:1, and 3:1;
Q _s	=	100 to 3,000 ppm; and
Manning's n	=	0.035.

In addition to these methods, regime equations, and permissible velocity can also be used if the channel geometry and velocity data are available. One method for design would be to combine a regime relationship or permissible velocity with the results of HEC-RAS.

HEC-6 has been suggested for use in siting and designing of grade control structures. The following is a brief history of the program and of the capabilities and limitations of applying the program to design and optimization of combinations of grade control structures. William A. Thomas at the Little Rock District, USACE, initiated HEC-6 development. HEC-6 evolved into Version 2.7 in 1976 during Mr. Thomas' employment at the USACE, Hydrologic Engineering Center (HEC). After leaving HEC, Mr. Thomas and his staff at WES developed the program into the network version of HEC-6 (sometimes referred to as TABS-1). In addition to network capability, several additional transport functions, a more complete computation of cohesive sediment suspension, and modification of the moveable bed width computation were added (USACE 1993).

In 1986, HEC released Version 3.2 as the Library Version until replaced by Version 4.0 in 1991. Version 4.0 included the WES network version, code upgraded to Fortran 77 Standard, miscellaneous changes to the program output, and minor error corrections. Because of these changes, Version 4.0 may produce different results than earlier versions. HEC released Version 4.1 in 1993. Version 4.1 includes sediment transport for grain sizes up to 2,048 mm, and some modification of data input capability. Version 4.1 of HEC-6 is currently available from the USACE, HEC website (USACE 2002).

In August 2000, the user's manual for HEC-6T, Version 5.13, was published by Thomas (2000), for the purpose of supplementing the 1993 HEC manual. A supplemental version from Thomas of HEC-6T was used for the computation of the Yalobusha Basin network, which included modification for the large tributary network.

Regardless of the HEC-6 version used, basic similarities and the purpose of the model remain constant. HEC-6 numerically simulates and predicts changes in river profiles resulting from scour and deposition over a period of time. Thomas (1995) listed characteristics of the program, as follows:

- one-dimensional;
- moveable boundary;
- steady-state open channel flow;
- continuous sequence of flows segmented into a series of steady flow events;
- step-backwater computation;
- sediment transport rates computed at each cross section;
- volumetric accounting of sediment in each reach;
- amount of scour or deposition computed for each reach and cross-section geometry is adjusted;
- sediment calculations are performed by grain size fraction; and
- allows for simulation of hydraulic sorting and armoring.

He also listed limitations to the program:

- one-dimensional;
- sequence of steady-state conditions;
- meandering not considered;
- formation of network computations is constrained:
 - sediment transport in distributaries is not possible,
 - flow around islands cannot be directly accommodated,
 - only one local inflow point can occur between any two cross sections;
- split flows are not provided; and
- supercritical flow is approximated by normal depth.

The 1993 manual (USACE 1993) provides a discussion of the theoretical basis for movable boundary calculations. Both manuals provide a wealth of information, including warnings and limitations. Two of these that pertain to modeling for design of grade controls are given below:

The bed change calculation on the HEC-6T program was developed for large rivers in which bed changes were gradual and hydrographs rose and fell slowly with respect to time. In the small channels where the model is being applied today, the bed often fluctuates wildly during the passing of a hydrograph. Sometimes sediment will deposit during the peak flows and then erode during subsequent low flows. Since there are no checks on slope stability, the bed can become very irregular... (p. 8-1, Thomas 2000)

Irregular cross-sectional shapes have been observed to cause severe problems in simulating the DEC streams.

This program was designed for non-cohesive sediment transport. Some very limited cohesive theory was added for special purposes as it might relate to non-cohesive transport. This code was never intended to model cohesive sediment transport exclusively. However it has been used on some successful applications involving cohesive sediments by carefully posing the questions and confirming the model to prototype data. (p. F-4, Thomas 2000)

The following paragraphs review a 1995 HEC-6 model analysis of the grade control structure by Thomas (1995). The purpose of Thomas' (1995) investigation was to develop a procedure for spacing grade control structures, and to include computed sediment continuity in that design process. The effort did not optimize the spacing of grade control structures, and only three conditions were analyzed: 1) the existing condition with DEC structures as constructed in 1995 using the 1986 to 1992 U.S. Geological Survey (USGS) gauging station data; 2) the without DEC condition using the existing condition morphology without structures and the same 1986 to 1992 hydrology as in condition 1; and 3) the future condition using the existing condition morphology (condition 1) and repeating the 1986 to 1993 USGS gauge hydrology four times to simulate a 30-year period of record. The existing condition model was run to confirm model performance. Results of the existing condition model compared well with field data points.

While no optimum design capability was developed, Thomas (1995) demonstrated a systematic procedure for analyzing the effects of grade control structures. Thomas (1995) recommended using SAM (Thomas *et al.* 1994) or empirical methods, such as slope-area

relationship (Harvey and Watson 1986) for preliminary spacing of structures. Final design calculations for performance of structures were recommended for evaluation with HEC-6.

While each of the design tools cover a wide range of application and capability for analysis, no single model is presently available for system-wide design of equilibrium that can be applied with relative ease. It is critical that a suitable procedure be developed.

CHAPTER 4 FIELD DATA COLLECTION AND ANALYSIS

Chapter 4 provides a collection of field-acquired data and preliminary analyses of sites, based on data availability at each site. Grade control structure sites are located in Mississippi, Illinois, Missouri, and Colorado. By comparing the various types and performance of the structures, a database has been developed to provide insight into developing a design criterion for siting and spacing control structures. The data allow consideration of a wide variety of structures and design approaches.

4.1 FIELD REVIEW SITES

The following data have been collected through review of available data and through field surveys conducted. Information for nine sites is presented:

- Burney Branch Creek, Mississippi
- Blue Creek, Illinois
- Blue River, Colorado
- Brush Creek, Kansas
- Hotopha Creek, Mississippi
- Little Snake River, Colorado
- South Fork of Little Snake River, Colorado
- Middle Worsham Creek, Mississippi
- Perry Creek, Mississippi

4.1.1 Burney Branch Creek, Mississippi

Burney Branch Creek is located in Oxford, Mississippi. The study reach is 6,000-ft long, extending downstream from twin concrete box culverts under Highway 7 (N34°20;12.06", W89°30;51.48"). Approximate watershed area at the downstream end of Burney Branch is 10 sq mi. Two rigid sheet pile and concrete, CIT-type drop structures have been utilized in rehabilitating this reach of Burney Branch. Both structures were constructed in 1982 and were designed to contain the 100-year discharge and include the provision for floodplain storage using valley dams in conjunction with each structure. Since 1984, several major channel-stabilization projects have been constructed upstream. A 1997 survey of the entire reach indicated some change in slope since 1993 (Figure 4.1).

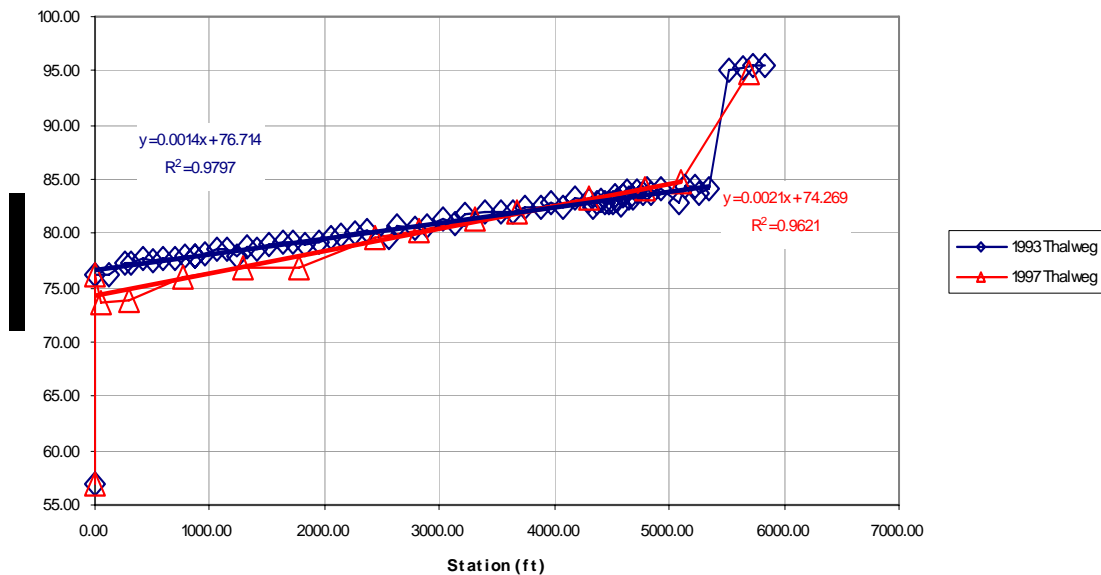


Figure 4.1: Comparison of Burney Branch thalweg profiles for 1993 and 1997

The design for spacing, vertical drop, and projected channel gradients was based on tractive stress analysis (Hayward and Daniel 1981). Original design of the structures provided for a bed slope of 0.0008 between structures. It is apparent that upstream sediment supply has been too great to allow the slope to adjust to the design slope. Table 4.1 is a listing of pertinent structure information.

Table 4.1: Burney Branch, Mississippi, structure locations and details

	Structure No.	
	1	2
Station (ft)	0	5,525
Crest Elevation (ft)	76.19	95.03
Bed Elevation (ft)	56.96	84.18
Vertical Drop between Structures (ft)		18.84
Distance between Structures (ft)		5,525

Figure 4.2 indicates that the specific stream power for the 2-year recurrence interval discharge is near or below the median (30 W/m^2) for up to 2,500 ft upstream of the grade control structure, and then increases to values 2 to 3 times the median value. The stream reach of relatively high values occurs as the stream narrows in an armored section of the study reach. At thalweg slopes shown of 0.0014 and 0.0021, significant thalweg change must occur to achieve the design slope of 0.0008. The surveyed slopes (1993 and 1997) can be attributed to significant sediment supply from upstream. In 2002, it was noted during field inspection that the degradation has exposed a utility line at about station 3000, and efforts to support this line have resulted in construction of a small drop. The structures are functioning as designed with no maintenance problems. Although no difficulty has been noted, with vertical drops in excess of 2 m, safety is a concern as well as the obvious impediment to fish passage.

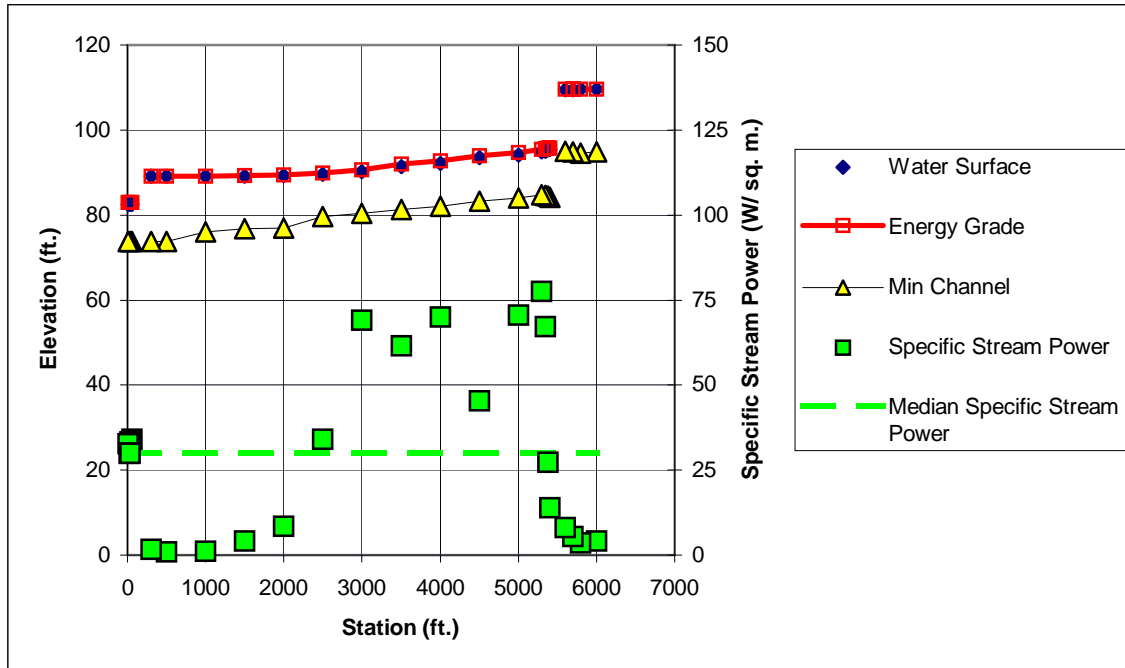


Figure 4.2: Hydraulic analysis of Burney Branch

4.1.2 Blue Creek, Illinois

Blue Creek is located in Pike County, Illinois approximately 5 mi outside of the town of Pittsfield, Illinois (N39°40.698', W90°46.368'). Headcutting of the channel resulted in severe erosion affecting the downstream water supply. Construction of a series of pool and riffle sequences in 1998 stabilized the channel and enhanced instream habitat. The study reach is 3,500-ft long, extending upstream from the County Road 2400 E. Bridge with an approximate watershed area of 3 sq mi. At the time of construction, the study reach thalweg slope was approximately 0.0038 (Figure 4.3). During the 2002 survey, the slope between structures averaged 0.0012 (Figure 4.4).

The rock weir structures (Figure 4.5) were selected as the appropriate feature to control incision and to be utilized in stabilizing the channel and enhancing instream habitat by Roseboom *et al.* (2000). Newbury and Gaboury (1993) suggest that naturally formed riffles occur about every 5 to 7 bankfull widths, and that the elevation of the weir crest should be determined to be at an elevation that does not cause a backwater at bankfull discharge. In 2002, the average spacing was found to be 284 ft; bankfull width for a discharge of 450 cfs varied from 24 to 52 ft. Average bankfull width was 32 ft. An average elevation difference between weir

crests was found to be 1.1 ft, and was relatively evenly distributed between weirs (Table 4.2). Pool depth increased an average of 0.5 ft during the period 1998 to 2002.

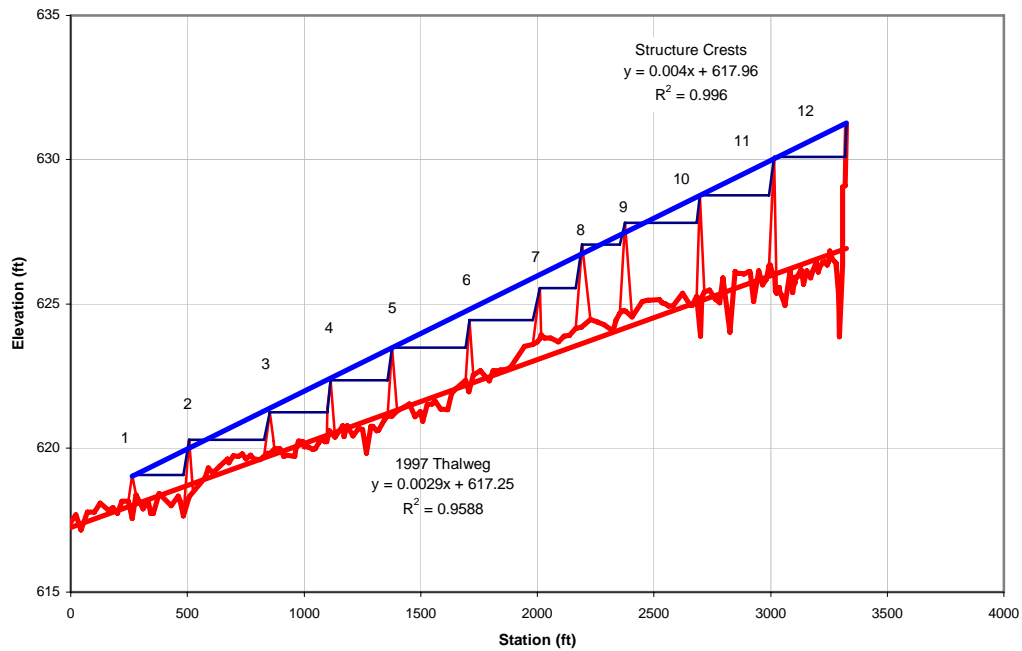


Figure 4.3: Blue Creek, Illinois, 1997 design thalweg profile

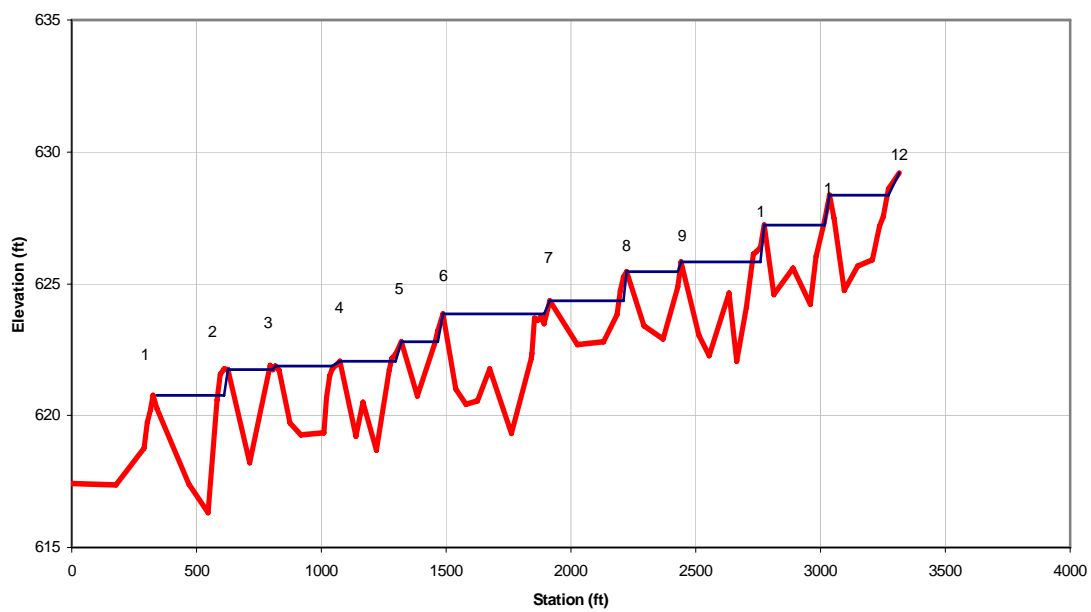


Figure 4.4: Blue Creek, Illinois, thalweg profile surveyed in 2002

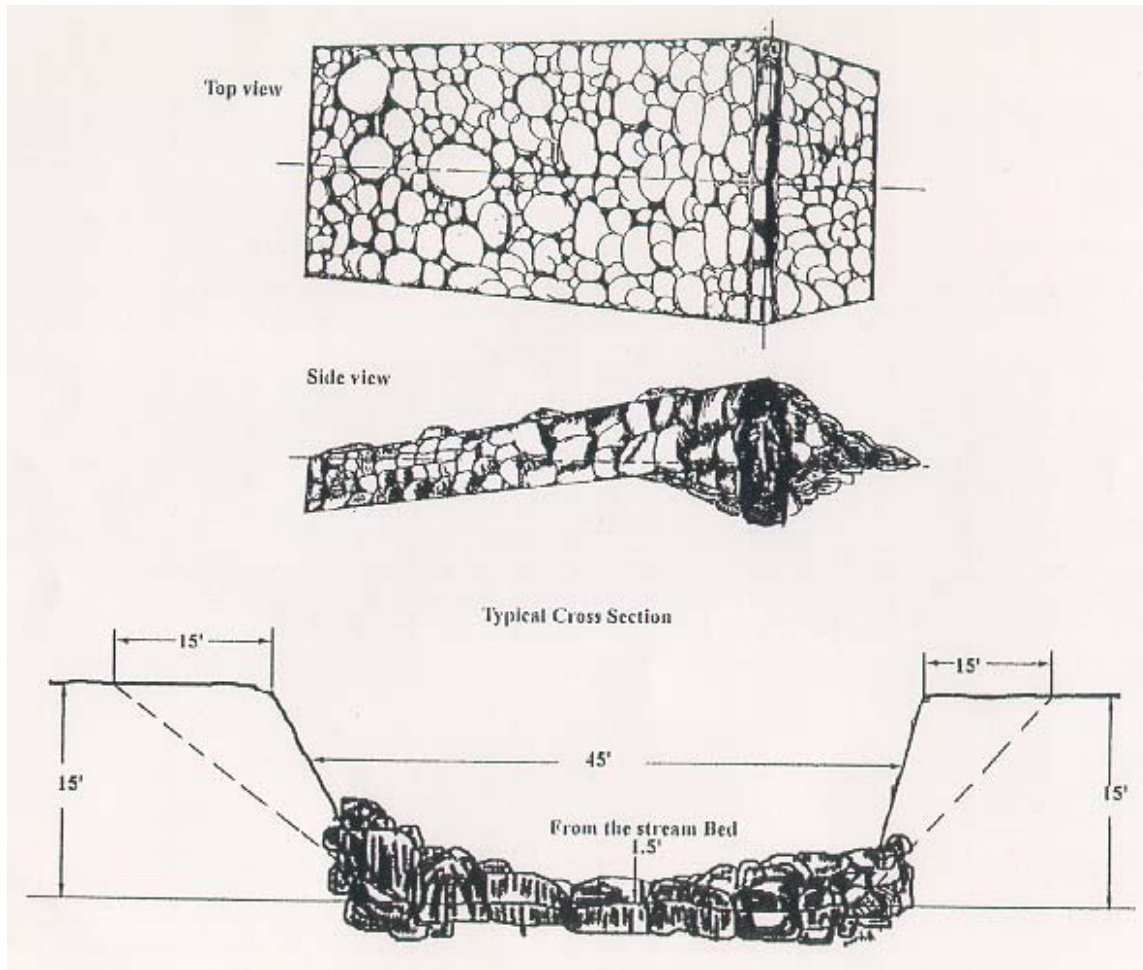


Figure 4.5: Blue Creek, Illinois, rock weir structure plans

The height of structures above the pre-construction bed varied from 2 to 5 ft. Crest stone diameters averaged 3 ft but the size of crest stones were highly variable. The downstream slope of each structure was 20:1, 20 times the height of the crest stone above the streambed. The upstream face of the weir extended upstream four times the height of the crest stone. Roseboom *et al.* (2000) stated that no additional stabilization efforts have been required since construction. The eroding stream banks have revegetated and the pools formed by each structure have remained open.

Table 4.2 lists pertinent structure information and shows a comparison of the structure design from 1997 and the current channel condition in 2002. Figure 4.6 is a photograph of a structure one month following construction. Figure 4.7 is the same structure 18 months following construction.

Table 4.2: Blue Creek, Illinois, structure comparison

Structure	Station (ft)	Spacing (ft)	2002 Survey Crest Elevation (ft)	2002 Elevation Difference Between Structures (ft)	1997 Design Crest Elevation (ft)	1997 Elevation Difference Between Structures (ft)	Difference in Crest Elevation (ft)	2002 Pool Depth (ft)	Design Pool Depth (ft)	Difference in Pool Depth (ft)
1	436	289	620.3	1.5	619.1	1.1	1.2	4.5	1.4	3.0
2	725	209	621.7	0.0	620.2	1.1	1.6	3.5	1.8	1.7
3	934	249	621.7	0.3	621.3	1.1	0.4	2.6	1.5	1.1
4	1182	252	622.1	0.7	622.4	1.1	-0.3	3.4	2.5	0.8
5	1434	170	622.8	1.0	623.5	1.1	-0.7	2.1	2.5	-0.5
6	1604	512	623.8	0.5	624.6	1.1	-0.7	4.5	2.1	2.4
7	2116	312	624.4	1.1	625.7	1.5	-1.3	1.7	1.9	-0.2
8	2428	246	625.5	0.4	627.2	0.7	-1.7	2.6	3.0	-0.4
9	2675	338	625.8	1.4	627.9	1.0	-2.1	3.8	3.3	0.5
10	3013	263	627.2	1.1	628.9	1.3	-1.7	3.0	4.8	-1.8
11	3276	282	628.4	0.8	630.1	1.3	-1.8	3.6	5.2	-1.6
12	3558		629.2		631.4		-2.2			



Figure 4.6: Blue Creek, Illinois, rock weir one month following construction



Figure 4.7: Blue Creek, Illinois, rock weir 18 months following construction

Relatively high values of specific stream power at the location of weir structures are shown in Figure 4.8. However, locations of high stream power are also locations at which the stream morphology is controlled by stone structures. The median value for the set of structures was found to be 18 W/m^2 . While each structure may be considered as a rather subtle feature, the combined effort provides energy dissipation and grade control allowing control for approximately 9 ft.

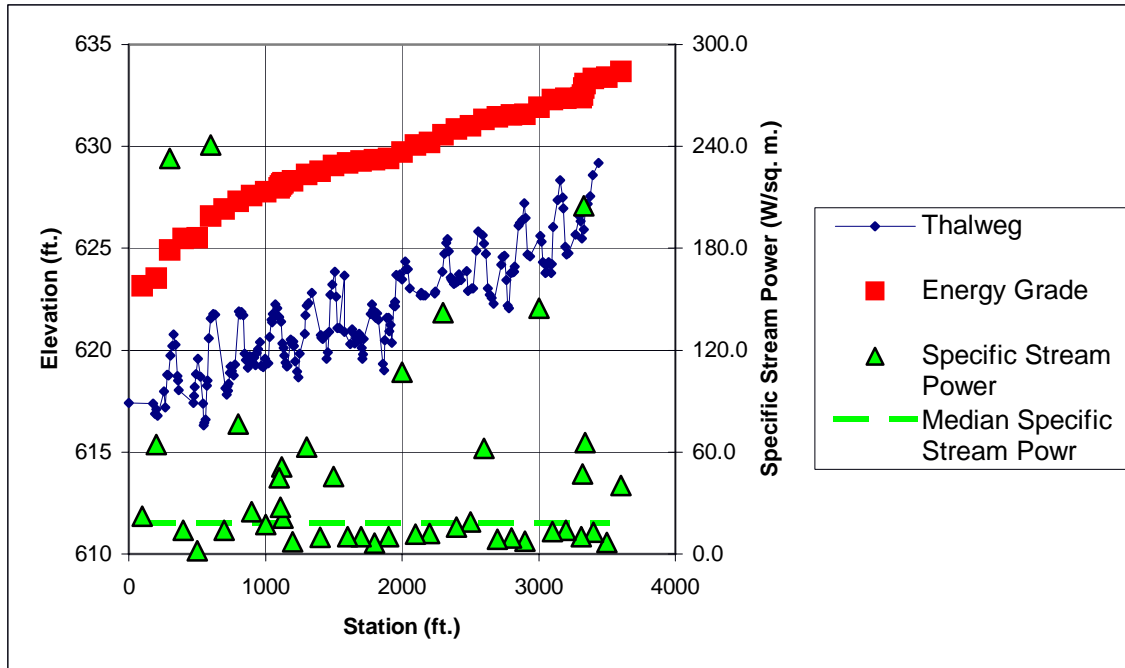


Figure 4.8: Hydraulic analysis of Blue Creek, Illinois, showing energy grade, thalweg elevation, and specific stream power

4.1.3 Blue River, Colorado

Blue River is located in Summit County in the town of Breckenridge, Colorado. During the late 1800s and early 1900s, 5 mi of the Blue River valley was mined by dredge boats. The mining resulted in an unstable river system with sparse riparian vegetation and many reaches in which surface flow disappeared for most of the year. As mitigation of destruction of existing wetlands, Breckenridge undertook the project to reclaim and create wetlands along a 1-mi section of the Blue River. The design was developed to greatly increase the duration of flows, stabilize the channel, and establish an environment capable of supporting wetland plant species. Components of the design included measures to stabilize the channel, and removal of dredge piles to create a configuration of channel and overbank areas offering the proper environment for wetland community establishment.

The study reach is approximately 8,000-ft long extending downstream from the Valley Brook Street Bridge (Figure 4.9) to the Coyne Valley Road Bridge (N39°51.589', W106°05.027'). Approximate watershed area at the downstream bridge is about 68 sq mi. The slope of the rehabilitation reach is shown in Figure 4.10 and is an average slope of 0.0147 for the

study site. A small tributary supplied by groundwater is approximately 850 ft in length, and is located at station 2800. The tributary was found to have an average slope of 0.01444 (Figure 4.11).

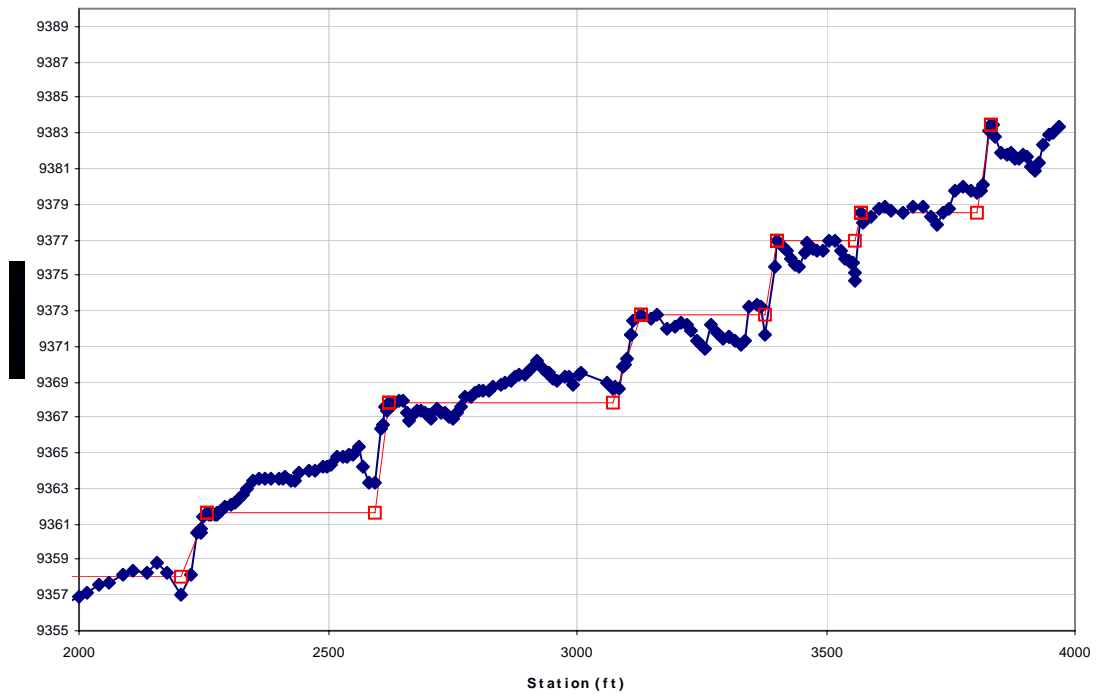


Figure 4.9: Blue River, Colorado, thalweg profile from station 2000 to station 4000

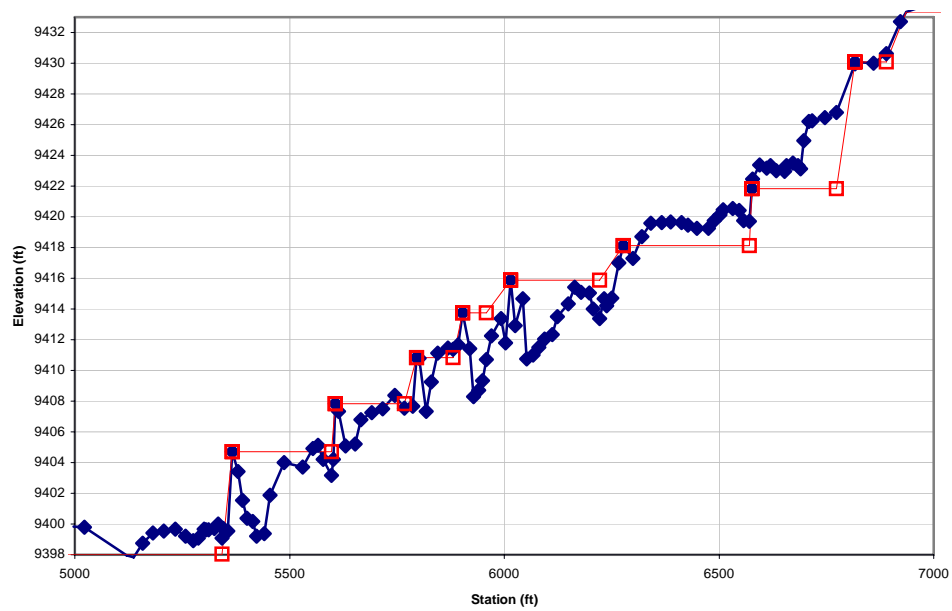


Figure 4.10: Blue River, Colorado, thalweg profile from station 5000 to station 7000

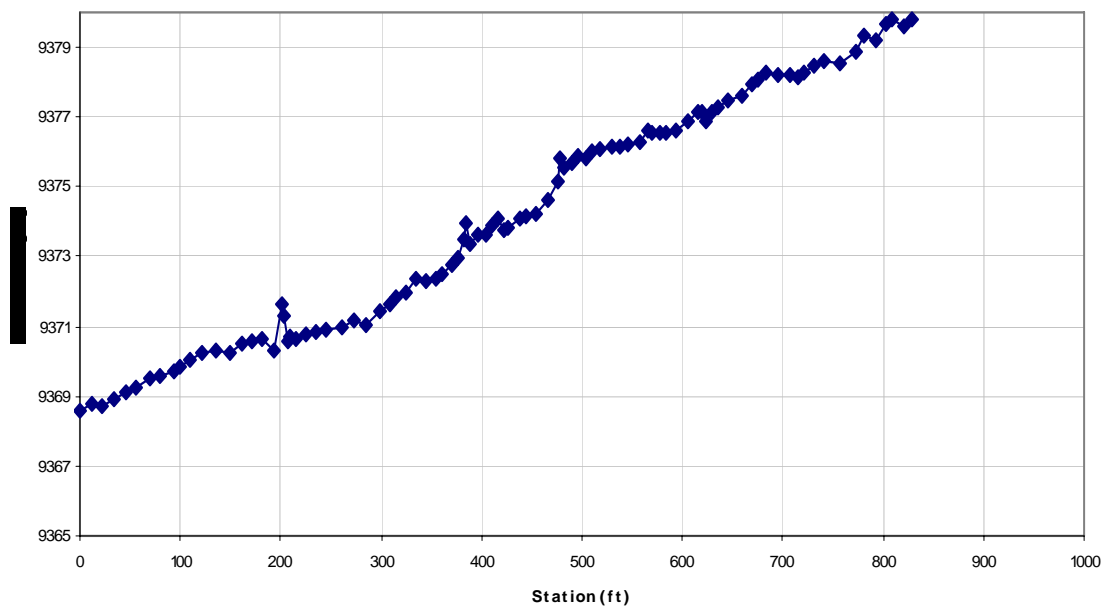


Figure 4.11: Blue River, Colorado, 2002 tributary thalweg profile

Design criteria for the channel restoration was conducted by Lenzotti and Fullerton Consulting Engineers, Inc. Channels with varying dimensions and bed slopes were considered and the resulting hydraulic conditions calculated for each configuration. An incipient motion analysis was conducted for both the bed and banks utilizing Shields' criteria as modified by Gessler (1971). From the analysis, it was determined that a channel with a width ranging between 30 to 40 ft and side slopes of 2.5:1 or flatter would be highly stable at an effective slope of 0.005 for flows up to the 25-year return period. To establish an effective slope of 0.005, a series of 21 drop structures with design heights of approximately 3 ft were included in the design. Sizing of the boulders for the drop structures was based on the factor of safety method presented by Simons and Sentürk (1992).

The configuration of the drop structures was designed to concentrate flows in the center with the crest being arched in the upstream direction and the center being slightly depressed. Therefore, the side slopes would not be exposed to the severe hydraulic conditions at the drop face. To dissipate energy and prevent downstream scour, a plunge pool was provided for each drop. At the downstream end of the plunge pool, large boulders were intermittently placed to protrude into the flow and function similar to baffle blocks. An apron of boulders ranging in diameter from 9 in. to 2 ft were placed 15 to 25 ft downstream of the plunge pool. To prevent flanking of the drop structure, bank protection continued for 15 ft upstream of the crest. In addition, the banks were graded to direct overbank flow into the channel upstream of the drop structures. All side slope protection was keyed 3 ft into the bed and an entire row of boulders was keyed below the plunge pool at the base of the drop face. Table 4.3 is a listing of pertinent structure information.

Table 4.3: Blue River, Colorado, structure locations and details

	Structure No.						
	1	2	3	4	5	6	7
Station (ft)	1,751	2,257	2,622	3,127	3,403	3,569	3,830
Crest Elevation (ft)	9,358	9,362	9,368	9,373	9,377	9,378	9,383
Vertical Drop between Structures (ft)		3.59	6.25	4.95	4.11	1.57	4.99
Distance between Structures (ft)		505	365	505	277	165	261
	8	9	10	11	12	13	14
Station (ft)	4,177	4,458	4,657	4,910	5,366	5,607	5,796
Crest Elevation (ft)	9,383	9,392	9,396	9,398	9,405	9,408	9,411
Vertical Drop between Structures (ft)	-0.85	8.90	4.41	2.09	6.64	3.15	3.02
Distance between Structures (ft)	347	282	198	253	456	241	189
	15	16	17	18	19	20	21
Station (ft)	5,903	6,015	6,277	6,576	6,817	6,937	7,477
Crest Elevation (ft)	9,414	9,416	9,418	9,422	9,430	9,433	9,441
Vertical Drop between Structures (ft)	2.91	2.11	2.24	3.74	8.24	3.23	8.05
Distance between Structures (ft)	107	112	262	299	241	120	540

Field discharge measurements during the relatively dry summer of 2002 showed that, at that time, the tributary contributed a large portion of the downstream flow. The tributary supplied 12.5 cfs and the main channel supplied 10.5 cfs. It was evident that the groundwater flow from this tributary (~13 cfs) was essential for habitat and stream flows in the downstream sections of the study reach. During low flows, the study reach upstream from the tributary had very little flow in the channel and the designed wetland areas outside the channel appear to have little to no water present at these low flows.

Figures 4.12 and 4.13 show the types of structures used within the study reach. Unfortunately, no hydrologic data were available for hydraulic analysis of this site. A few of the structures were found in need of maintenance.



Figure 4.12: Typical Blue River, Colorado, drop structure



Figure 4.13: Typical Blue River, Colorado, sloping drop structure

4.1.4 Brush Creek, Kansas

Brush Creek is located in Johnson County in the town of Mission Hills, Kansas. The watershed is completely urbanized and the channel is actively incising. Bed material is clay and poor quality sandstone. Channel width varies due to adjacent land use, but averages approximately 200 ft. The study reach is approximately 1,500 ft in length. As shown in Figure 4.14, the bed profile of Brush Creek is very irregular. The deeper portions of the surveyed profile

coincide with locations of unstable banks and undermining of the landscape rock walls that form the channel banks. A series of five rock weir structures were used to control the incising channel within the study reach.

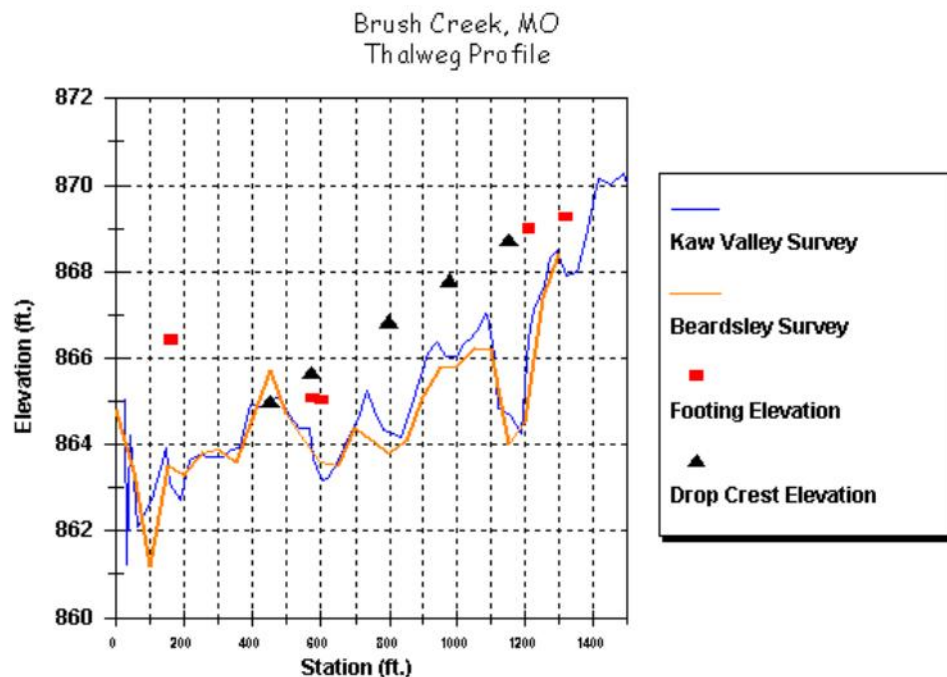


Figure 4.14: Brush Creek, Kansas, thalweg profile

The structure design followed the concepts set by Newbury and Gaboury (1993), which included construction of natural stone riprap, and made to appear as natural riffle features. The riprap was sized using both USACE (1991) recommendations and using the Abt and Johnson (1991) equation.

The downstream slope of the rock structure is a critical factor in rock sizing, and a slope of 1:20 (5%) was used. Steeper slopes would require larger sized rock, and steeper slopes could be considered a potential safety hazard in urban settings. The recommended gradation, computed using the Abt equation required a D_{50} of approximately 2 ft, with a D_{90} of about 3 ft, and a D_{10} of about 1.1 ft. Placement thickness was twice the D_{50} , or about 4 ft, with a minimum thickness of the D_{100} stone, or about 3.2 ft. Each structure was constructed by placing the riprap gradation upstream and downstream of a structure crest constructed of large quarry stone. Each crest stone was in excess of 1 m in length and 0.67 m in thickness. The riprap upstream of the

crest was placed at the angle of repose, and on a 5% slope downstream of the crest. Inspection of the site following a major flood that was in excess of the 100-year recurrence interval event suggest that the rock structures were adequately sized.

Table 4.4 includes the structure number, the approximate width of the structure at the crest, the crest elevation, and the bed elevation.

Table 4.4: Brush Creek, Kansas, structure details

	Structure No.				
	1	2	3	4	5
Station (ft)	450	575	800	975	1,150
Crest Elevation (ft)	865	865.67	866.67	867.8	868.73
Bed Elevation (ft)	865.7	864.1	863.8	865.8	864
Vertical Drop between Structures (ft)		0.67	1.2	0.93	0.93
Distance between Structures (ft)		125	225	175	175
Structure Width (ft)	54	55	41	35	30

Median specific stream power for Brush Creek is shown in Figure 4.15 as 58 W/m^2 for an estimated bankfull discharge of 1,000 cfs. This urban stream is highly confined by rock and concrete walls. Subsurface infrastructure and adjacent private landscaping limited the flexibility to widen the stream. In addition, the design included consideration of urban flood control for the 100-year recurrence interval discharge, a regulatory flood event (Federal Emergency Management Agency (FEMA)). Design calculations were required to demonstrate the elevation of the regulatory event was not exceeded.

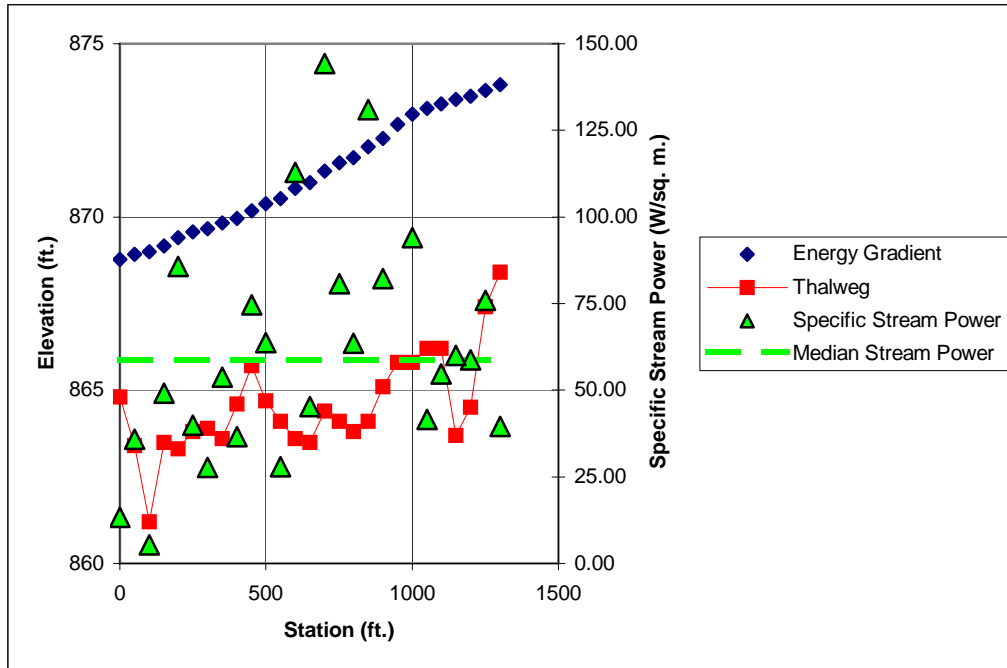


Figure 4.15: Hydraulic analysis of Brush Creek, Kansas

4.1.5 Hotopha Creek, Mississippi

Hotopha Creek is located in Panola County (N34°19.295', W89°47.336'). Hotopha Creek is 8,000-ft long, extending 7,000 ft upstream and 1,000 ft downstream from the Highway 315 Bridge (Figure 3.20). Approximate watershed area of Hotopha Creek is 35 sq mi. Hotopha Creek was channelized in 1961, and was initially surveyed by the Vicksburg District, USACE, in 1985. Subsequent surveys were conducted by CSU with 1995 being the most current. The bed slope from the 1995 survey is shown in Figure 4.16.

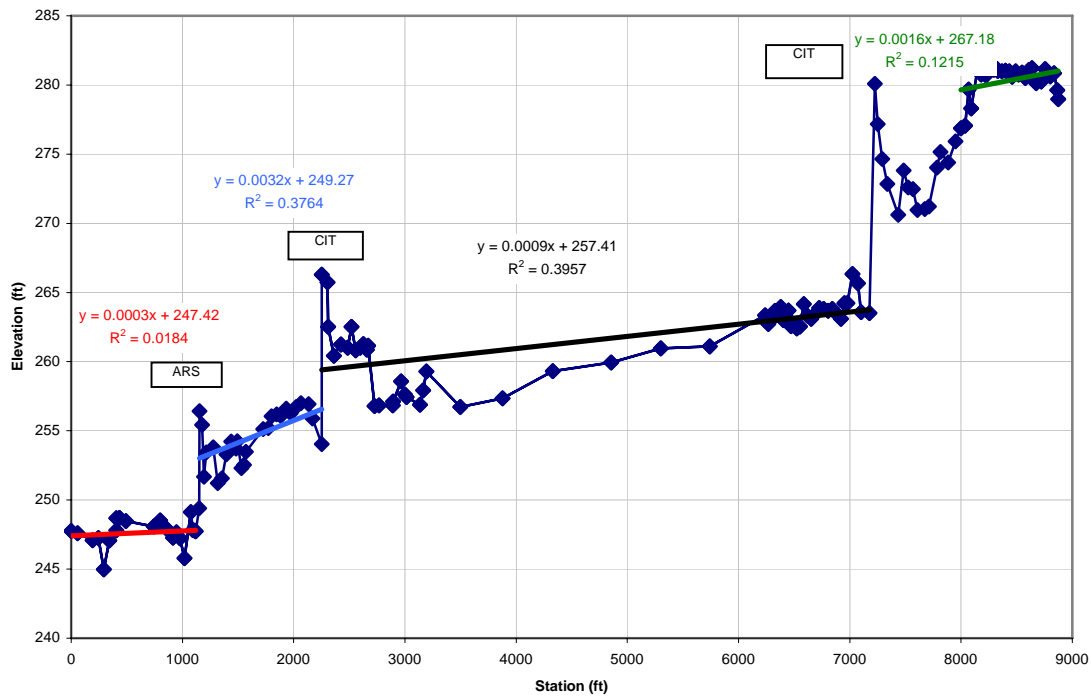


Figure 4.16: Hotopha Creek, Mississippi, 1995 thalweg profile

The structure design for spacing, vertical drop, and projected channel gradients was based on equilibrium slope. Original design of the rigid CIT-type structures provided for a bed slope of 0.0015 between structures. The bed slope between the ARS-type structure and the first CIT-type structure is 0.0032, which is generally considered to be unstable. Upstream of the high drop the slope averages 0.0009, however, a portion of the thalweg has not filled. In similar fashion, upstream of the second high drop had not filled in the 1995 survey and the slope of the portion of the thalweg shown (upstream of Marcum Creek tributary) is filling at a slope of 0.0016. Table 4.5 is a listing of pertinent structure information. Figure 4.17 shows the high-drop structures used in this study reach.

Table 4.5: Hotopha Creek, Mississippi, structure details

	Structure No.		
	1	2	3
Station (ft)	1,100	2,250	7,200
Crest Elevation (ft)	257.5	266.2	280.8
Bed Elevation (ft)	247.7	254	263.5
Vertical Drop between Structures (ft)		8.7	14.6
Distance between Structures (ft)		1,150	4,950



Figure 4.17: Hotopha Creek, Mississippi, CIT-type structure

Median specific stream power for the entire study reach is 31 W/m^2 , and the points significantly above the median are located in the short reach downstream of the downstream high drop (Figure 4.18). Field observation indicates greater stability with the reach controlled by high drops; however, some of the downstream instability may be, in part, caused by sediment storage upstream of the high drops.

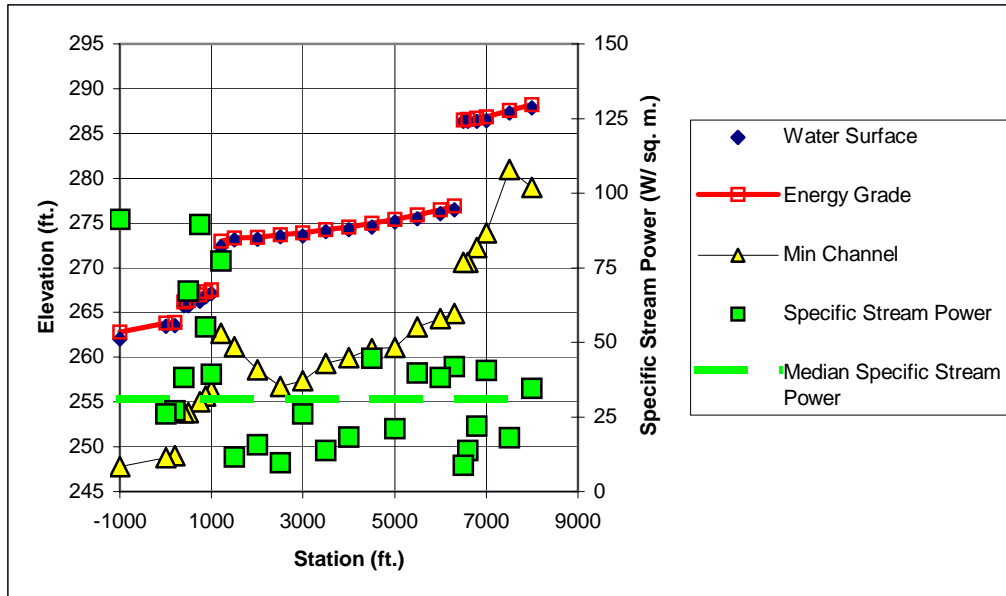


Figure 4.18: Hydraulic analysis of Hotopha Creek

4.1.6 Little Snake River, Colorado

Little Snake River is located in Routt County approximately 30 mi north of Steamboat Springs, Colorado. The study reach is 12,000-ft long extending 8,200 ft upstream from the confluence (N40°59.222', W107°03.079') on the South Fork and includes 3,700 ft of the main channel (Figure 4.19). The slope of the rehabilitation reach on the South Fork is an average of 0.0082. Portions of the South Fork survey are shown in Figures 4.20 and 4.21. The slope of the rehabilitation reach on the Little Snake River is shown in Figure 4.21 and has an average slope of 0.005 for the study site.

The structure design for spacing, vertical drop, and projected channel gradients was based on field determinations by Rosgen (2002a). A variation of cross-vane, J-hook vane, and boulder cluster structures were used in the channel-restoration process. Tables 4.6 and 4.7 provide details of the structures surveyed, and Figures 4.22 through 4.26 are photographs of the structures.